

PROCEEDINGS THE INSTITUTION OF CIVIL ENGINEERS

PART I
MARCH 1954

ORDINARY MEETING

17 November, 1953

WILFRID PHILIP SHEPHERD-BARRON, M.C., T.D., President
in the Chair

The Council reported that they had recently transferred to the class of

Members

BERRIDGE, PHILIP ROGER SETON, B.Sc. (<i>Eng.</i>) (<i>London</i>).	BURROWS, ALAN.
BOURNE, NIGEL BRIDGES, M.A. (<i>Cantab.</i>).	GORDON, LESLIE, B.Sc. (<i>Edinburgh</i>).
BRESLIN, THOMAS.	LLEWELLYN, WILLIAM JAMES.
	RUTHERFORD, WILLIAM, B.Sc. (<i>Durham</i>).

and had admitted as

Graduates

ABERNETHY, JAMES, B.Sc. (<i>Aberdeen</i>), Stud.I.C.E.	BLASCOS, PETER MANUEL, B.Sc. (<i>Nott- ingham</i>).
ALBINSON, BRIAN, M.Sc.Tech. (<i>Man- chester</i>), Stud.I.C.E.	BREACH, RONALD BERTRAM, Stud.I.C.E.
ALLISON, HUGH BRUCE, Stud.I.C.E.	BURTON, ROBERT ERNEST, B.Sc.(<i>Eng.</i>) (<i>London</i>), Stud.I.C.E.
ATAK, DOUGLAS TASSO, B.Eng. (<i>Sheffield</i>), Stud.I.C.E.	COLTON, JOHN BAINES, Stud.I.C.E.
BARLOW, DAVID MORTIMER, Stud.I.C.E.	COOK, BRIAN ARTHUR, B.Sc.Tech. (<i>Man- chester</i>), Stud.I.C.E.
BARNETT, NORMAN NATHANIEL, Stud. I.C.E.	COPE, Geoffrey Hulme, B.Sc.(<i>Eng.</i>) (<i>London</i>), Stud.I.C.E.
BEAK, WILLIAM ROBERT JENNER, B.Sc. (<i>Eng.</i>) (<i>London</i>), Stud.I.C.E.	CORBETT, BRIAN OLIVER, B.Sc. (<i>Man- chester</i>), Stud.I.C.E.
BERNARD, PETER, B.Sc.(<i>Eng.</i>) (<i>London</i>), Stud.I.C.E.	COWBOURNE, DENNIS STUART, B.Sc. (<i>Eng.</i>) (<i>London</i>), Stud.I.C.E.
BERWICK, DONALD BRUCE, B.Sc. (<i>Edin- burgh</i>), Stud.I.C.E.	CRAIG, JAMES SIDNEY, B.Eng. (<i>Liverpool</i>), Stud.I.C.E.
BIRD, FRANK HARRY, B.Sc.Tech. (<i>Man- chester</i>).	CROCOMBE, COLIN ALSTAN, B.E. (<i>New Zealand</i>), Stud.I.C.E.

- CROOK, DOUGLAS McFEAT HEWITT, B.Sc. (*Glasgow*), Stud.I.C.E.
 DAVIES, JOHN DAVID LENÈGRE, B.Sc. (Eng.) (*London*), Stud.I.C.E.
 DAVIS, BRIAN JOHN, Stud.I.C.E.
 DONOVAN, RONALD, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 DREWETT, GEORGE IAIN HAY.
 DUNLOP, DENNIS STEWART, B.Sc. (*Belfast*).
 DUNTON, DAVID ANTHONY, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 EDMONDS, JOHN KERRIDGE, B.A. (*Oxon.*), Stud.I.C.E.
 EVELEGH, GEORGE FREDERIC ROSSLYN NIAL, B.A. (*Cantab.*).
 FAITH, JOHN, B.Sc. (*Belfast*).
 FAYE, JAN TROYE, B.Sc. (*St Andrews*).
 FISHEY, DAVID HOWARD, B.Sc. (*Wales*).
 FRANCE, ALAN JOHN STUART, B.Sc. (*Birmingham*), Stud.I.C.E.
 GAHAN, DERMOT FRANCIS, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
 GOUGH, JOHN NOEL, B.Sc. (*Birmingham*), Stud.I.C.E.
 GRAY, DERRICK JOSEPH, B.Sc. (*Durham*).
 GRIEVESON, MARK BRIAN, Stud.I.C.E.
 HARRISON, JOHN GEOFFREY, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 HART, MAXWELL BRYCE, B.E. (*New Zealand*).
 HILL, JOHN BRIAN, B.Sc. (*Wales*).
 HILLS, ROGER PAUL, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 HINDE, GORDON, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 HIRSCHORN, NORMAN, B.Sc.(Eng.) (*London*).
 HODGKINSON, IAN VICTOR, Stud.I.C.E.
 HOGG, ROBERT CAMPBELL, Stud.I.C.E.
 HOWARD, ROSS KENRICK, B.E. (*New Zealand*), Stud.I.C.E.
 HOYLAND, GEORGE ANTHONY, B.Eng. (*Sheffield*), Stud.I.C.E.
 HUGHES, BARRY PETER, B.Sc. (*Birmingham*), Stud.I.C.E.
 HULLY, LESLIE, B.Sc. (*Belfast*).
 JENKINS, ROYSTON VIVIAN, B.Sc. (*Wales*), Stud.I.C.E.
 JOBLING, DAVID GEOFFREY, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 JONES, DANIEL EDWARD REECE, B.Sc. (Eng.) (*London*).
 JONES, JOHN BRYNMOR, B.Eng. (*Liverpool*), Stud.I.C.E.
 JONES, PETER HAROLD CHARLES, B.Eng. (*Sheffield*).
 KATHIRAMALAINATHAN, KATHIRGANAR VEERAGATHY SINNATHURAL, B.Sc. (Eng.) (*London*).
 KENNEWELL, JOHN GEOFFREY, Stud. I.C.E.
 KENNY, BERNARD JOHN, B.Sc. (*Belfast*).
 KING, DAVID LEONARD, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 KIRKALDY, IAN DRUMMOND, Stud.I.C.E.
 KRAMER, ALEXANDER ERIC, B.Sc. (*Aberdeen*).
 LITTLE, BRIAN GEORGE, Stud.I.C.E.
 LONGBOTTOM, DEREK PATTERSON, B.Sc. (*Durham*), Stud.I.C.E.
 LUMB, PETER, M.Sc. (Eng.) (*London*), Stud.I.C.E.
 MCCURDY, JOHN ALEXANDER IRWIN, B.Sc. (*Belfast*).
 MACKAY, WILLIAM KENNETH, B.Sc. (*Glasgow*), Stud.I.C.E.
 McKENNA, BRIAN ROBERTSON, B.Sc. (Eng.) (*London*), Stud.I.C.E.
 McKERRACHER, IAN, Stud.I.C.E.
 MACKIE, GEORGE ARTHUR, B.Sc. (*Glasgow*), Stud.I.C.E.
 McMILLAN, THOMAS BROWNEIGG, B.Sc. (*Glasgow*).
 MAGUIRE, DENNIS PETER, B.Sc. (*Bristol*).
 MAHALINGAM, SIVAPRAGASAM.
 MARLEY, PAUL COUCH, Stud.I.C.E.
 MATHERS, ROBERT CHARLES.
 MEREDITH, JOHN HOWARD, Stud.I.C.E.
 MISEROY, NORMAN HENRY THIRSK, Stud.I.C.E.
 OSBELDISTON, KENNETH, B.Sc. (*Nottingham*), Stud.I.C.E.
 OSWELL, JOSEPH RIDLEY, Stud.I.C.E.
 PAGE, LESLIE ROY, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 PASK, DAVID ALLEN, B.Eng. (*Sheffield*).
 PERERA, KORUWAKANKANAMALAGE DON PIYADASA, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 PRICE, SELWYN LIONEL, B.Sc. (*Witwatersrand*).
 PUGH, CHRISTOPHER, B.Sc. (*Bristol*), Stud.I.C.E.
 RAMSBOTHAM, IAN THOMAS BRIDGE, B.Eng. (*Liverpool*), Stud.I.C.E.
 RANDERSON, MALCOLM DAVID, B.Sc. (Eng.) (*London*), Stud.I.C.E.
 RHODES, JOHN ARTHUR, B.Sc. (*Durham*), Stud.I.C.E.
 ROGERS, ERIC, B.Sc. (*Aberdeen*).
 ROSS, VINCENT ANTHONY, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 RUSSELL, KENNETH SIDNEY, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 RUSSELL, SAMUEL OLIVER, B.Sc. (*Belfast*).
 RUTTLEY, HAYDN, B.Sc. (Eng.) (*London*).
 SALTMARSH, DAVID ARTHUR, B.A. (*Cantab.*).
 SCANNELL, JOHN ANTHONY, B.E., B.Sc. (*National*), Stud.I.C.E.
 SCOTT, DAVID BUCHANAN, Stud. I.C.E.
 SEAR, KENNETH ALLAN, Stud.I.C.E.

- SHAW, KENNETH WILLIAM, B.Sc. (*Cape Town*).
 SIMKINS, ALAN, B.A. (*Cantab*), Stud. I.C.E.
 SKINNER, JOHN, B.Sc. (Eng.) (*London*), Stud.I.C.E.
 STAMMERS, ALBERT JAMES, Stud.I.C.E.
 STENING, LESLIE GASTON, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 STEPHENS, PETER DONALD, B.Sc. (*Bristol*), Stud.I.C.E.
 STONE, MICHAEL DENNIS, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 TAIT, ROBERT, B.Sc. (*Aberdeen*).
 TAVENER-SMITH, NORMAN WILSON, B.Eng. (*Liverpool*).
 TAYLOR, HARRY, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 TAYLOR, JAMES WILLIAM HENRY, B.Sc. (Eng.) (*London*).
 TEBBUTT, RONALD NORMAN, B.Sc. (*Birmingham*), Stud.I.C.E.
 THOMAS, AUSTIN WITHIEL, B.E. (*New Zealand*), Stud.I.C.E.
 THOMAS, DAVID HENRY WILLIAM, B.Sc. (Eng.) (*London*), Stud.I.C.E.
 THOMAS, JAMES HAROLD, B.Sc.(Eng.) (*London*).
 THOMPSON, WILLIAM HENRY, Stud.I.C.E.
 TURNER, FREDERICK HENRY, B.Sc. (Eng.) (*London*), Stud.I.C.E.
 VAN SOMMERS, JOHN CHARLES, Stud. I.C.E.
 Warburton, MICHAEL STANLEY, Stud. I.C.E.
 WATT, ALAN JOHN, B.E. (*New Zealand*), Stud.I.C.E.
 WATT, JAMES CHALMERS GRANT, B.E. (*New Zealand*).
 WELLS, THOMAS NEVILLE, Stud.I.C.E.
 WILLIAMS, DAVID LONZIL, B.Sc.(Eng.) (*London*), Stud.I.C.E.
 WILSON, BRIAN HAROLD, B.A. (*Cantab.*), Stud.I.C.E.
 WRIGHT, PETER JOSEPH FREDERICK, B.Sc. (Eng.) (*London*), Stud.I.C.E.

and had admitted as

Students

- ADDYMAN, JOHN FRANCIS.
 AUGHTON, JOHN RYCROFT.
 BAILEY, JOHN.
 BERRY, NORMAN STEVENSON McLEAN.
 BLINKHORN, HUGH JOHN.
 BRUNTON, JOHN DAVID.
 CLELAND, WILLIAM BAIRD WILSON.
 COOK, ARTHUR ERNEST.
 COOK, DAVID ALLEN.
 DAVIS, REGINALD MAURICE.
 DEAR, NEIL DOUGLAS GAUNTLETT.
 DE ZOYSA, LALITHE NIHAL.
 DOUGLAS, RODNEY HUGH ROVERY.
 DREW, GEORGE BRIAN.
 DYE, RONALD ALFRED.
 EVANS, RONALD THOMAS.
 FEATHERSTONE, RONALD EDGAR.
 FOX, MALLOCH RICHARD CLAYTON.
 GHAZAL, MUDHAFFAR SALIH.
 GIBBS, BRIAN SHEPPERSON.
 GILHAM, DAVID RONALD.
 GLICK, DAVID HARVEY.
 GOONEWARDENA, EGODAGE LINCOLN EDWARD KALANATIS.
 GREEN, FRANCIS EDWARD.
 HALL, GORDON WARD.
 HARRIS, JOHN EVANS.
 HEDLEY, ROBERT GORDON.
 HEPTONSTALL, BARRIE THOMAS.
 HEWES, ROBERT HARRY.
 HO THIAN HOCK.
 HOBBS, JOHN.
 HOLLOWAY, JOHN McKINSTRY.
 HOSSACK, JOHN FREDERICK.
 HOTCHKISS, KENNETH ALAN.
 HUNTER, KENNETH.
 INWANG, ISAIAH ISAAC.
 IRWIN, MERVYN.
 IRWIN, MICHAEL JOHN.
 JESS, ANTHONY.
 JOEL, JEREMY CHRISTOPHER.
 JONES, DAVID KEITH.
 JONES, PETER GORDON.
 JONES, RONALD HUMPHREY.
 KEOGH, GERALD SEAN.
 KINDER, GRAHAM.
 KOLL, GODFREY ERNEST.
 KUCIEWICZ, JERZY MIKOLAJ.
 LA CHAPELLE, SYDNEY OSCAR RAMÓN.
 LARREA-PEREZ, JULIAN.
 LINDSAY, PAUL CHARLES.
 LOUGHNAN, ANTHONY O'BRIEN.
 McKIBBIN, WILLIAM IVAN.
 McNICHOLAS, MICHAEL.
 McROSTIE, ALEXANDER.
 MARGASON, GEOFFREY.
 MILES, GEORGE EDWARD.
 MITCHELL, ROBERT BURTON.
 MORRICE, BERNARD JOHN.
 MURRAY, WILLIAM DAVID CHARLES.
 NAVARATNAM, KANDAVANAM.
 NICHOLLS, ROBERT.
 PARKER, NORMAN ALAN.
 PARRY, BRIAN.

PATIENCE, JOHN ANTONY.
PENNY, HUGH ALAN LAIRD.
PETHIYAGODA, EKANAYAKA DUNUKARA
MUDIYANSELAGE LEELARATNE.
PITHER, MAURICE JOHN.
RAMMELL, JOHN CHRISTOPHER.
RATHMELL, BERNARD TIMOTHY.
RIDINGS, IAN DAVID.
RUSH, MICHAEL ALLEN FRANK.
SHARP, BRIAN NEVILLE.
SMITH, DAVID HARRY.

SMITH, MICHAEL ERNEST.
SMITH, PETER GRAHAM.
STEGGALL, MALCOLM RONALD.
STRACHAN, MICHAEL.
STRATFORD, JOHN EDWIN.
SYKES, JOHN KINGSTON.
THOMPSON, ALFRED.
TROWMAN, DERRICK IVOR.
WELLS, FRANCIS REGINALD.
WILLIAMS, JOHN GRAHAM.
WILLIS, EDWIN JOSEPH.

The following Paper was presented for discussion and, on the motion of President, the thanks of the Institution were accorded to the Authors.

Paper No. 5971

**“The Construction of Two Concrete-Lined Tunnels for the
Machkund Hydro-Electric Project, South India ”**

by

George Alexander Gauld, B.Sc., M.I.C.E.

and

Roger John Newport, B.Sc., A.M.I.C.E.

SYNOPSIS

The Paper describes the construction of two concrete-lined tunnels in rock for the Machkund Hydro-Electric Project, South India.

Modern techniques were adopted and emphasis is given to the use of modern plant and equipment and to the way in which unskilled eastern labour was successfully trained to use it.

The use of light rock-drills mounted on air-legs as the basic method of driving is described in detail together with the problems encountered and overcome in using a pneumatic placer for the linings.

A description of the construction of a surge shaft and pipe tunnel associated with the general tunnel works is also included.

INTRODUCTION

THE construction of concrete-lined tunnels in rock is no new development in South India ; several exist which have been in use for many years. Most of them, however, were constructed with the minimum of plant using hand labour so far as possible, the emphasis being laid upon the immediate economy of low labour rates rather than speed of construction.

Post-war conditions called for a different approach, and for the Machkund Tunnels, the contractors decided to take full advantage of the latest plant and technique available.

Constructional work of this nature suffers from two handicaps—the usually isolated and remote locations of the work sites, and the absence of skilled plant operators and constructional workers. On the one hand there is the desire to mechanize as much as possible but, on the other hand, there are difficulties to face both in getting the plant to the site and keeping it running, and in finding and training labour to operate it in a country which is, as yet, far from being industrially developed.

The equipment and methods used, together with the results obtained, are described in the following pages.

GENERAL DESCRIPTION OF THE PROJECT

The Machkund Hydro-Electric Project is located about 130 miles (by road), north-west of the South Indian port of Vizagapatam, in undeveloped hill country on the Machkund river which, in this area, forms the boundary between the States of Madras and Orissa. The Project is sponsored jointly by these two States and was inspired by the Duduma Falls where the Machkund plunges 540 feet into a deep gorge.

Although jointly owned, it is being constructed under the direction of the Madras Government Electricity Department which will also operate and maintain it. The power will be supplied to each of the two States through separate transmission lines originating at the power house. Preliminary investigations were carried out as long ago as 1929, when the exploration parties came in by elephant transport from the Vizagapatam-Jeypore road 12 miles away. Work on the access road to open the site was started in 1946.

The lay-out of the Project is shown in *Figs 1*. A diversion dam is located some distance above the falls to take advantage of the additional head given by the rapids which lie above the falls. The water passes along an open flume channel, then through the flume tunnel, 4,110 feet long, and discharges into the tunnel pond formed by a small masonry dam across the Makkiput stream. Across the stream valley, opposite, and about 30 feet below the flume tunnel exit, the water enters the pressure tunnel. This terminates in a surge shaft followed by a short pipe tunnel, giving a total tunnel length of 3,090 feet.

From the surge shaft, two 10-foot-diameter pipes pass through the pipe tunnel and, at the first anchorage, each splits into three 6-foot-diameter pipes which take the water down the steep side of the gorge to the power house located on the river bank, giving an effective working head of 840 feet.

In the first development, only one set of three pipes is being installed. The second set will be completed at a later stage together with the Jalaput dam now under construction about 14 miles above the falls. This dam will create a storage capacity of about 25,650 million cubic feet. On completion, the Project will have an installed capacity of 100,000 kilowatts for consumption in the towns and industrialized areas of the coastal plain where a grid system is already established.

The sides of the Machkund gorge are covered by tropical jungle but at the higher levels where most of the works are situated, it is mainly open grassland and scrub. Shade temperatures range from 40° F. at night during the winter to 115° F. in the hot weather prior to the monsoon. The average rainfall is 55 inches and falls for the most part during the months June to October. Two or three flood periods of concentrated

rainfall lasting from a few hours to a day or two can, however, be expected. The area is malarial.

ROADS, CAMPS, AND SERVICES

The Madras Electricity Department assumed responsibility for constructing access roads and camps, and the provision of water, electric power, medical, and other services. The major works were let out on contract and all the contractors availed themselves of these facilities.

The labour camps were located in two main centres: one near the diversion dam and the other roughly half-way between the tunnel pond dam and the exit of the pressure tunnel.

Large centralized camps show economies in the provision of services, but when shared by several contractors, administrative difficulties arise. Furthermore, in tunnelling, where men work throughout the day and night, the problem of transport has to be met. Men will not walk long distances in the heavy monsoon rains, nor will they take the risk of encountering wild animals at night. In these circumstances, smaller camps located close to the work sites would appear to offer many advantages.

Since the main Department power station closed down for several hours on Sundays for maintenance, small generator sets were provided to light the tunnels for tunnel-maintenance work during those periods. They also served for emergencies.

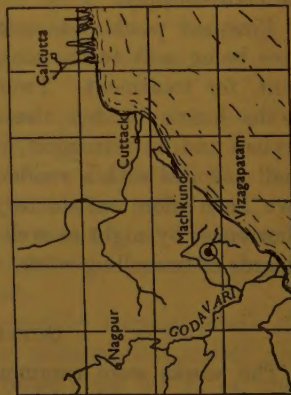
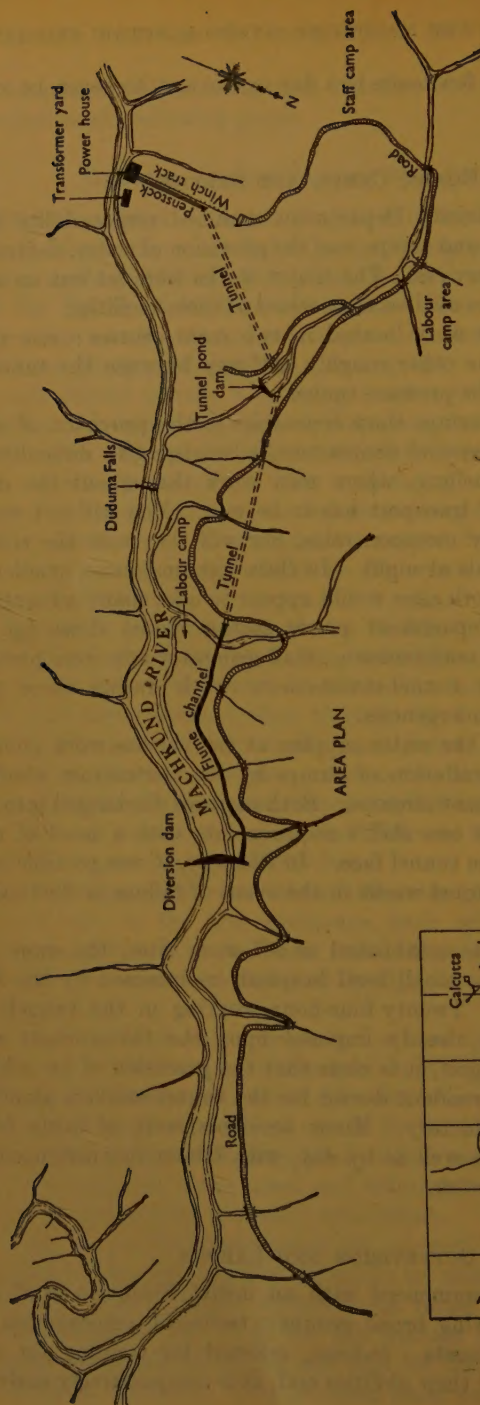
So far as possible, the water supplies at the various work points were duplicated by the installation of pumps by the contractors which could draw water from adjacent streams. Both supplies discharged into storage tanks holding at least one shift's requirements with a head of not less than 100 feet above the tunnel face. In this way, it was possible to avoid interruptions to the tunnel works in the event of failure or fluctuations in the mains supply.

First-aid posts were established at all work sites, the more serious cases being sent to the small local hospitals maintained by the Department, for treatment. Twenty-four-hour working in the tunnels added to the heavy burden already imposed upon the Government medical services and, in retrospect, it is clear that the provision of an additional small hospital with a resident doctor for the tunnel workers alone would have been more satisfactory. Minor accidents were of fairly frequent occurrence, by night as well as by day, with labour inexperienced in the hazards of tunnelling work.

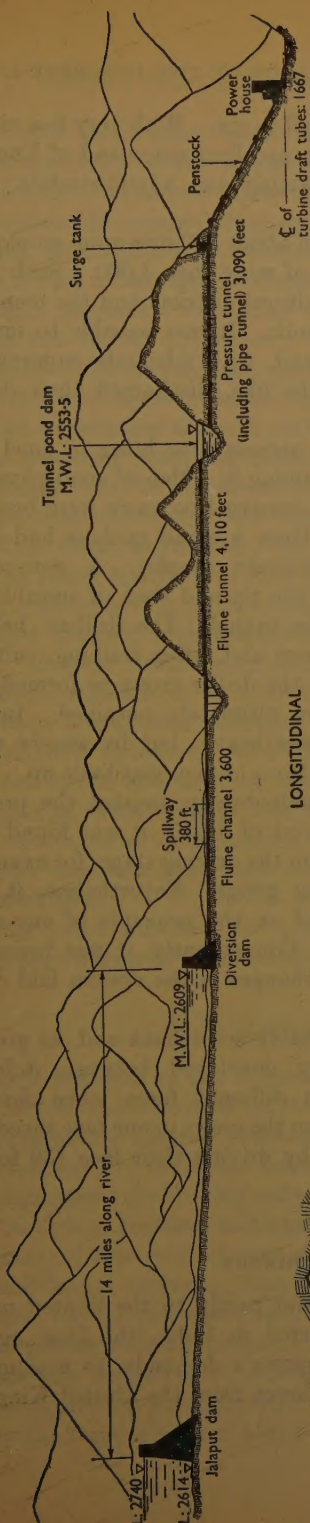
SUPERVISION AND LABOUR

The works were commenced with an initial European staff of ten, divided into the following broad groups: technical, construction, plant maintenance, and accounts. Indians, selected for training for responsible posts soon proved their abilities and, at a comparatively early stage,

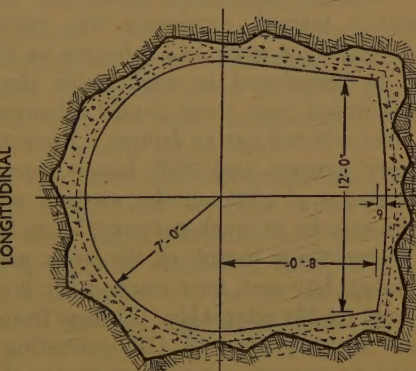
Figs 1 (a)



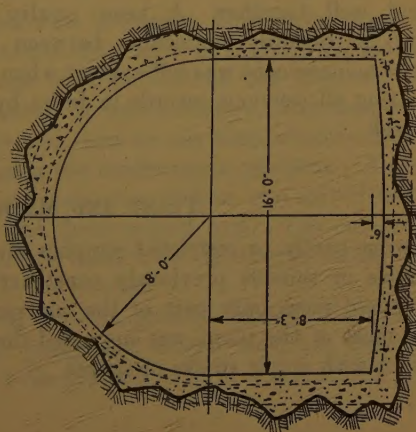
Figs 1 (b)



LONGITUDINAL



PRESSURE TUNNEL



FLUME TUNNEL

SECTIONS

it was possible to reduce the European staff to six. With very few exceptions, the supervisory Indian staff had an excellent command of English, and this fact eased the problem of training and administration very considerably.

During most of the construction period about 800 men were employed, with a peak strength, for a few months, of more than 1,000. Such local labour as was available came from agricultural districts and the men had little or no experience of construction work. It was possible to import tradesmen from a reasonable distance and, with adequate supervision, these carpenters, masons, fitters, and the like, discharged their duties remarkably well.

For the work in the tunnels, it was necessary to bring in good raw material and to face up to the task of training it. Men of good physique and having some previous experience of construction work were brought in, often from considerable distances. Once a small nucleus had been adequately trained, expansion was relatively simple. A new-comer would commence work as a labourer on the tip and then, if suitable, he would be transferred to work inside the tunnel. The drillers' helpers were recruited from these tunnel labourers and, after training, suitable men were selected out as drillers. From the driller group so formed, the necessary gangers and shift bosses were ultimately obtained. In the earlier stages of training, the wastage was heavy, but by steady work and the exercise of much patience, a fine team of men was built up.

The repetitive nature of the work undoubtedly assisted the process of training, but once men reached the level of driller, it was found that they were readily adaptable. Drillers from the driving stage, for example, became the key men in the concreting gangs. Nevertheless, it was essential to operate a rigid system based on the principle of one man, one job, and deviations led to trouble. Consequently, it was necessary to plan ahead with great care to avoid changes in the system laid down for any new operation.

Individually, the men became remarkably self-reliant and, as groups, worked well together. A keen rivalry developed between different groups, particularly noticeable between different faces when driving. Pride of achievement was self-evident when the crews in one face succeeded in beating all previous records in India by driving their face 130 feet in one week.

PLANT AND MACHINERY

Owing partly to restricted supplies and partly to the limited use of machines in tunnels previously constructed in India, the local agency firms could meet only part of the contractor's demands for equipment. Thus, most of the plant was imported direct from the United Kingdom together with ample stocks of spares.

Import restrictions, customs formalities, and the task of bringing heavy plant up to the site over indifferent roads, including a difficult *ghat* section, introduced many unforeseen delays. Nevertheless, the task was eventually accomplished, and once the problem of training the maintenance crews had been overcome, the plant met the demands placed upon it with a high degree of reliability.

Much ingenuity was called for in the manufacture of unobtainable spares and special equipment; construction work of this nature located far away from commercial centres, requires the provision of comprehensive workshop facilities at the site.

The importance of good maintenance cannot be too highly stressed. With relatively unskilled labour, lacking a true mechanical sense, it was found to be impossible to leave routine maintenance in the hands of the machine drivers. Their responsibilities were limited to keeping the machines clean, to running them, and to the reporting of faults.

The plant was divided into groups: rockdrills, compressors, locomotives, rocker-shovels, pumps, crushers, etc., and maintenance gangs were specially trained, each responsible for servicing one group of machines only. Routine duties were laid down for shift maintenance, with an additional schedule to be carried out weekly on Sundays.

When breakdowns occurred, local attention was prohibited. In the case of fixed equipment, such as compressors, a trained repair crew was sent out to the site to effect repairs. Light machines, such as rockdrills and pumps, were brought into the central workshops where they could receive attention under proper supervision. So far as possible, a few serviced machines were kept ready for exchange to minimize delays at the work sites.

This policy increased the maintenance-labour costs but was justified in the high degree of plant-reliability so obtained.

METHOD OF DRIVING

Since the ends of the tunnels were accessible, both were driven from each end without the need for intermediate adits or shafts. Furthermore, the programme allowed sufficient time for driving the pressure tunnel first, followed by the flume tunnel. It was necessary, therefore, to provide equipment for driving only two faces at one time, thereby reducing the total plant required and easing the maintenance problem. A central compressor-house located near the tunnel-pond-dam site served first the pressure-tunnel entrance, then the flume-tunnel exit. The compressors provided for the pressure-tunnel-exit drive were transferred to the flume-tunnel entrance on completion of the former.

The two tunnels were driven through granitic gneiss. Soft patches were encountered together with jointy and broken rock, but much of it was in the form of a hard blue close-grained rock, in which ordinary steel

drill-bits were blunted by about 12 inches of drilling. This led to the original decision to use tungsten-carbide bits, and both detachable bits and tipped steels were actually employed.

As an alternative to using a drill carriage with mounted drifters, a plan for using only light water-fed machines on air-legs with a suspended platform for drilling the upper holes permitting simultaneous drilling and mucking, was finally adopted. The sequence of operations (see *Figs 2*) was as follows :—

As soon as the fumes from the blast had cleared, part of the drilling gang proceeded to the face and trimmed off all loose rock. The remainder of the drilling gang brought in the drilling equipment and set this up on the muckpile at the face for drilling the middle band of holes. At the same time, the mucking gang brought in two rocker shovels which were put to work at the toe of the muckpile which usually extended to about 50 or 60 feet from the face.

By the time the drillers had completed all the holes which could be reached comfortably from the muckpile, the latter had been shortened to about 15 feet. At this point, both drilling and mucking were stopped while all hands helped to erect the suspended platform, a task which usually occupied about half-an-hour.

The suspended platform was carried on two girders, each made up from a pair of 2-inch pipes with angle bracing welded in between. Pairs of holes were drilled into the tunnel walls on each side to correspond with the spacing of the open ends of the pipes. Thus, steels could be inserted, half into the holes and half into the ends of the pipes, forming four supports to each of the girders. The required holes were drilled from the muckpile, one girder being erected close to the face, and the other about 8 feet back. A timber deck was added, and on this the drillers completed the top holes while the rocker-shovels completed the mucking underneath. (See *Fig. 10.**)

As soon as the mucking was finished, the drillers started on the lower holes which could be drilled from the floor. At the same time, the mucking gang removed their equipment, made the necessary extensions to the tracks, cleared drains, and assisted the shot-firer in preparing the explosives.

On completion of the drilling, the face was charged with explosives, the suspended platform being used for the upper holes. This was then dismantled, the tunnel cleared, and the round fired, completing the cycle. Average times for these operations are given in Appendix I.

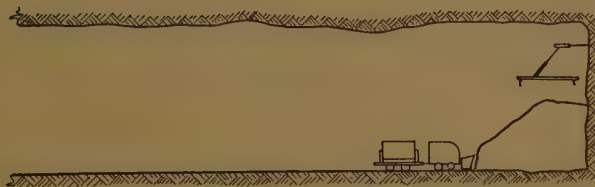
CRITICISMS OF THE METHOD

Early fears that the use of air-legs with inexperienced labour would result in poor alignment of the drill holes were unfounded. The men

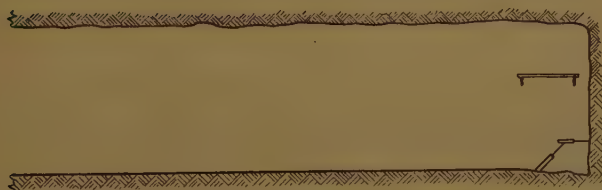
* *Figs 10 to 17* are all photographs and are printed between pp. 130 and 131.—
SEC. I.C.E.

Figs 2

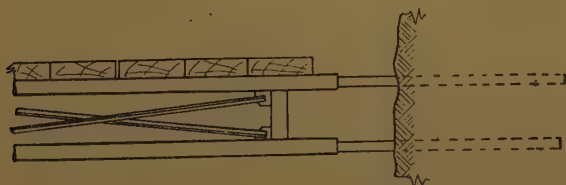
Mucking and drilling middle holes



Mucking under platform whilst drilling upper holes



Drilling lower holes



Method of supporting ends of platform

DRIVING TECHNIQUE

soon appreciated the need for accuracy in position and direction, and once a hole was started, usually with some help from the face ganger, the driller could be trusted to maintain the required alignment.

The burn-cut holes called for the greatest accuracy, but a simple routine was developed. The first hole was completed with a 10-foot steel. The machine was withdrawn, the steel being left in the hole. A 5-foot steel was fitted and the next hole started. The first 10-foot steel was then drawn out a few feet and thus provided a directional guide for completing the second and subsequent holes.

Thus, the rockdrill-and-airleg combination was entirely satisfactory from the point of view of directional control, and offered the advantages of light weight, easy handling, and quick changes from one position to another. It was found, however, that the air-legs were easily damaged, and care was necessary when handling them.

Hardened steel bars, of $1\frac{1}{4}$ inch diameter, were used to carry the ends of the platform girders. They frequently bent and jammed under the weight and added considerably to the task of dismantling the platforms. Bars of greater diameter would have called for special drill-bits and larger girder-pipes, but the weight of the girders was already as great as could be manhandled.

In broken rock, it was often difficult to drill holes in the tunnel walls in the required positions and in rock sound enough to take the weight. Overbreakage introduced an excessive gap between the pipe-girder ends and the tunnel walls, and while a telescopic end to the girders might have been devised, this would only have added further to the weight. It was usual in these circumstances to reduce the number of drillers on the platform and to add props, if necessary, although these interfered with the mucking.

Thus, the time taken for the erection and dismantling of the suspended platforms often considerably exceeded the average. With the attendant interruptions to drilling and mucking, the frequent delays had serious repercussions on the weekly footage.

In good rock these disadvantages were minimized, and the advantages of simultaneous drilling and mucking were realized. In comparison with full-face drilling, the suspended-platform technique reduced the equivalent drilling time to the sum of the time required to erect and dismantle the platform and the time required to drill only the lower holes where seven machines were employed. The "muckpile" and "platform" holes were drilled with five machines and these, together with the two rocker-shovels gave a lower peak-demand for compressed air than would any form of full-face drilling technique giving similar overall cycle-times. Thus, the suspended-platform method showed an economy in both compressor capacity and the total number of rockdrills required which, in these works, was a considerable asset.

VENTILATION : FUMES AND FOG

The exhausts from the compressed-air machines normally supplied sufficient fresh air for men working at the tunnel face, and the main purpose of ventilation was to remove the fumes arising from the blast as quickly as possible.

Since sheet steel was virtually unobtainable, 8-foot-by-4-foot semi-hard aluminium sheets were supplied for ducting. These were rolled and lap-jointed at site to form circular ducts about 15 inches diameter. Slip-in joints were used from duct to duct by making the diameter at one end slightly larger than the other. Two-inch-wide "insulating" tape was employed to wrap round and seal the joints, but since this was not entirely satisfactory, the joints received an additional wrapping of strips of hessian cloth soaked in a bituminous compound and put on hot.

The ducting was connected to a 10-horse-power electrically driven centrifugal fan located at the portal, equipped with reversing ducts. An axial-propellor-type air-driven booster-fan, operating in one direction only, was introduced into each line between the face and portal after the face was about 750 feet in.

Many experiments were carried out to determine the best method of clearing the blast fumes quickly, but no entirely satisfactory solution was found. The procedure commonly adopted was to keep the ducting up to within 200 feet of the face; to keep it nearer resulted in excessive damage from blasting. After the blast, the maximum supply of compressed air was admitted through the open-ended main, but restricted at a main-line valve ahead of the booster fan, partially closed prior to blasting. This valve maintained sufficient back-pressure in the line to operate the booster fan which was arranged to extract foul air from the face. The portal fan was operated on suction continuously, the reversing ducts not being used. The booster fan was shut off when drilling and mucking commenced, to conserve compressed air for the demands of the face.

A study of the cycle-times in Appendix I shows that the average time which elapsed between the blast and the point at which it was possible for the tunnel crews to resume work, formed an appreciable portion of the cycle-time. This was not a result of the men's reluctance to go to work. Indeed, it was frequently necessary to restrain them, for cases of gassing were not uncommon. The conclusion is that the ventilation system was inadequate for this size of tunnel.

The booster fans were extremely effective but, being air driven, called for air power at the time when there was a maximum demand on the compressors for blowing-out at the face. Thus, in spite of the cost of running in power cable, electric booster-fans would appear to be justified. Larger ducts would have been cumbersome, but improved results could be expected by doubling the lines and their corresponding fans.

The face work suffered from another handicap—fog. The humidity of the outside air was normally high, even during the "cold weather"

season, on account of the river and the tropical vegetation in the valley. Thus, warm saturated compressed air passed into the tunnel mains; its expansion through the machine exhausts and its consequent cooling created a thick fog. It is doubtful if the fog had any harmful effect, although it was thought to hold and delay the settling of dust, thus increasing throat irritation. But it had a strong psychological influence on the men, who believed it to be harmful.

Far more important, however, was its effect on the efficiency of the drillers. The most powerful floodlights could not penetrate the fog and, as soon as the five or seven machines went to work, no driller could see what was happening to his drill steel at the rock face. This tended to lead to a lack of alignment between the hole and drill steel, straining the latter and the machine chuck, and aggravating the causes of shank breakage.

On a similar location, the provision of after-coolers complete with water separators at the compressors to eliminate excessive water vapour in the compressed-air lines would, therefore, appear to be well worth while.

MUCKING

The cost of bringing in a normal type of mechanical excavator for use in the tunnel open cuttings alone was prohibitive, and no such machine was available elsewhere on the Project. Thus, the tunnel rocker-shovels were used for this work but their value was limited where rock was encountered. In open-cut work, where the use of explosives must be restricted for both physical and economic reasons, the rock tends to break into pieces too large for rocker-shovels to handle, and the work is delayed by the need for further drilling and blasting to reduce these to a manageable size. While the availability of the rocker-shovels was an advantage against the alternative of hand mucking in the open cuttings, the work would have been completed in much less time had it been possible to use a standard excavator.

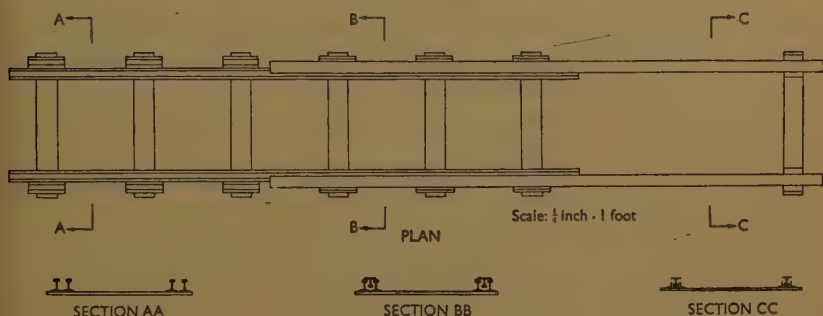
The tunnels were both of adequate width to allow the simultaneous operation of two air-driven rocker-shovels working on parallel 2-foot-gauge tracks. Diesel locomotives, pulling trains of up to eight 1-cubic-yard tip-wagons, were employed to bring the tunnel muck out to the spoil tips. Provided the locomotives were adequately maintained, particularly with regard to fuel pumps and atomizers, little trouble was experienced with their exhaust fumes.

The loose clamp-fittings for slide rails as suggested by most manufacturers gave constant trouble. Although maintenance was fairly heavy, slide rails of the type shown in *Figs 3*, which were evolved at site, proved to be quite satisfactory in service.

In the earlier stages of the job, full sets of scissors points and crossings connecting the two parallel tracks were used for changing over the wagons

at the back of the shovels. It was a major task to move them forward as the face advanced and in order to avoid delays to the face gangs two of the shifts were brought out on Sundays to perform this work. Not only did this add directly to the tunnelling-labour costs but, in being limited to a

Figs 3



DETAILS OF SLIDING RAILS

move only once a week, by the end of the week the lead from the rocker-shovels to the scissors crossings had become excessive for manhandling the wagons. It is never easy to push loaded wagons round curves, and the double reverse curves of the scissors crossings proved to be no exception. This added to the delays which interfered with the quick change-over of wagons needed to keep the shovels working effectively.

For the second tunnel, a simple, site-made wagon slide (see *Fig. 11*) was introduced in place of the scissors-crossing unit. It eliminated curved track and speeded up the handling of the wagons. Furthermore, it was easy to move forward and the move could be made by the regular mucking gangs in the interval between the completion of mucking at the face and finishing the drilling of the bottom group of holes. The wagon slide was moved forward every two rounds. This kept the leads to the minimum and eliminated the need for bringing out special gangs on Sundays for this work.

A study of the cycle-times shows that mucking was a decisive factor in the progress obtained. It is easy to be critical in retrospect but the lesson learned on these tunnels was that ordinary 1-cubic-yard tip-wagons are both too small and too light for operation with the larger-sized rocker-shovels if the full capacity of the latter is to be utilized.

Studies showed that even after wagon slides were installed, the shovels were actually loading during only about half the available time, the balance being taken up in changing the wagons behind the shovels. A marked reduction in mucking times would have been obtained by the use of wagons of at least double the capacity.

The wagons available were of the ordinary type with the skips made up in $\frac{1}{8}$ -inch steel plate. The shock of loading these with rock from rocker-shovels was considerable. Not only was the skip plating seriously punctured, but the skips soon became badly mis-shapen, causing trouble and delay when dumping the spoil on the tips. This led, indirectly, to distortion of the chassis and needless derailments. Maintenance was heavy and costly and frequently reduced the number of wagons available to considerably less than actual requirements. These troubles would have been very much reduced by the employment of heavier wagons suitably reinforced and built to withstand the rough treatment associated with rocker-shovel loading in rock tunnels.

DRILL STEELS AND BITS

The detachable bits used were of the cruciform type, whereas the tipped steels were of chisel form. In good rock, both forms of bits worked well, but in broken and jointy rock, the cruciform bit gave less trouble, the chisel form having a greater tendency to jam.

In comparing the overall performance of the two types, the figures given in Appendix II show that the tipped steels were superior to the drill rods with detachable bits. They had other practical advantages, but in reading the notes which follow, it must be appreciated that the only material available for the manufacture of drill rods was a poor-quality carbon-steel, whereas the tipped steels were made up in a high-quality alloy-steel.

With the tipped steels, there were far fewer breakages of the steels either at the shank or tip end, resulting in fewer interruptions to drilling.

Occasionally, a tipped steel was lost by becoming jammed in a hole beyond the possibility of extraction. The drill rods with detachable bits jammed less frequently, but gave much more trouble in that the bit itself was lost through rod breakage or broken by the bursting of the bit at the tapered connexion to the drill rod. Magnetic and other extractors were used, sometimes with success; but more often the hole, with its lost bit was abandoned, necessitating a new hole alongside, and so delaying the round.

The tipped steels, with few exceptions, outlasted the life of the tungsten-carbide tips. Thus, the tipped-steel sets, not having suffered re-shanking and shortening, were all of standard length. The holes were therefore drilled to their correct depth without difficulty, giving a square face and maximum "pull." The percentage of broken shanks was so low that it is doubtful if the provision of a sharpener and furnace is justified when using tipped steels. No labour or machines are required for forming taper or other connexions as are necessary between the rod and detachable bits.

For the upper holes where the muckpile or platform space was limited, the holes were normally drilled with two rods or steels, the first, 5 feet,

and the second, 10 feet long. For the lower holes, drilled from the floor, it was possible to drill the entire depth with one 10-foot tipped steel, thus cutting the drilling time by eliminating the loss of time in changing steels. The 10-foot drill-rods with detachable bits would not stand up to this treatment.

Since it was inadvisable to issue more than a slight excess over net requirements of detachable bits, when shanks or taper ends broke—as they frequently did—the bit, if not lost, would be changed over. It was therefore difficult to keep control over the usage of the detachable bits and they were often excessively worn when returned for regrinding. With the tipped steels, it was possible to exercise a better control and the steels were returned for regrinding after drilling a definite footage. This tended to increase the life of the steels.

Most, but not all, of the trouble experienced with the detachable bits was attributable to the drill rods with which they were used. Had they been made of steel comparable in quality with that of the tipped steels, it is probable that the detachable bits would have given a much improved performance.

BLASTING AND SAFETY PRECAUTIONS

In the interests of safety, 50-volt lighting was used at the tunnel faces. When charging the holes, floodlights were employed, being kept as far back from the face as practicable, all leads being kept strung up on the tunnel walls and off the wet floor.

In the earlier stages of the job, the contractor's chief engineer held a "blasting school" on Sunday mornings, at which the men were taught safety precautions, the routine to be followed, how to deal with misfires, and so forth. As an incentive, men who were passed out from the "school" were given a special certificate, and promotion beyond a certain grade was dependent upon a man holding this certificate. The rules were rigidly enforced and any breach of regulations resulted in suspension or dismissal.

The care taken was well justified for, quite apart from the economical gain in successful blasts resulting from this careful tuition, the driving of the tunnels was completed without a major accident; only one man was unfortunate enough to lose his life when dealing with a misfire in a way contrary to regulations.

A standard burn-cut type of round was used throughout in preference to other patterns to ensure that the muckpile would be short, high, and close to the face to facilitate the drilling of the middle holes from the muckpile.

The size of the charge made it imperative to operate the exploder outside the portal, and a heavy T.R.S. cable was permanently slung along the tunnel wall, opposite to that used for the lighting cables, to connect the exploder with the lead wire to the detonators at the face. A rigid system of testing the circuit was employed, and the actual blast was the

responsibility of the shot-firer. He held the exploder key and was personally responsible for clearing the tunnel of all personnel prior to blasting.

An added safety precaution was the provision of miners' safety helmets to all tunnel workers. In several minor falls of rock which did occur, these helmets undoubtedly saved a number of lives.

SECTIONS REQUIRING SPECIAL METHODS

The pressure tunnel, driven first, was in fairly sound rock throughout, and required no form of temporary support.

In the drive from the flume-tunnel entrance, a section of the line coincided with a rock formation consisting of heavily broken and jointed material lying between two shear planes slightly inclined from the vertical. A major roof fall occurred at one point, and for about 70 feet thereafter support was required. A section of soft rock interlaced with hard rock "boulders" was also encountered in the drive from the exit end of the tunnel, and special methods had to be adopted over a length of 108 feet.

Largely on account of limited steel supplies, and partly because the few Indians who had had any previous experience of soft-ground tunnelling gained in the construction of masonry-lined railway tunnels knew only the "English" method, a modification of this system was adopted. A length of about 8 feet was opened, using timber and light rails as was necessary for a temporary roof support. Ribs and laggings were then set up to a profile approximately 9 inches clear of the finished lining profile, and an 8-foot section of walls and arch concreted by hand to a rough finish. Light-rail arch-ribs were incorporated over the roof area as reinforcement.

The method employed readily available local materials and the men soon dropped into the new routine. Cycle-times are given in Appendix III.

A short section in very broken ground at the exit was constructed using conventional steel ribs fabricated from steel joists which were available.

PREPARATIONS FOR THE CONCRETE LINING

As soon as driving was completed, preparations for cleaning out the invert and placing a preliminary concrete mat were put in hand.

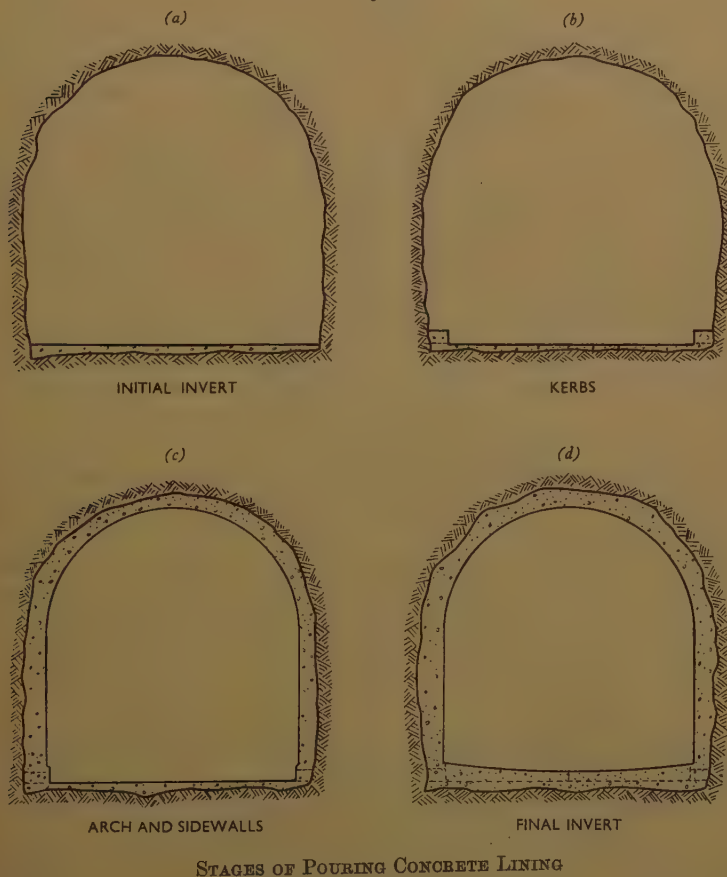
With the original muck track in the middle and with the men strung out, it was possible to clean out quickly the loose filling material on each side. This cleaning was carried out to the tunnel gradient to maintain natural drainage.

When this work was completed, the men were divided into two gangs. One gang worked ahead cleaning out the remainder of the invert across the full width, the muck being run out on the track ahead. This gang also carried out any necessary trimming of high rock in the floor and on the sides to a height of 4-5 feet.

The second gang followed, thoroughly cleaning off the bed-rock with

air and water jets. Next, they set precast concrete blocks, one series to carry a central 2-foot-gauge track, and another series, accurately set to level, for supporting screeds. Timber screeds were set in the blocks, and the initial floor concrete was laid (see *Figs 4*) using tip-wagons run in from the portal where the mixer was set up.

Figs 4



A small third gang followed, setting in the kerbs for locating the side-wall shutter skirts, to an accurate line.

The final trimming was carried out by a fourth gang. The trimming gantry was mounted to run on two sets of standard 2-foot-gauge track and fitted with two profiles. That at the leading end was fixed, and cut to a clearance of 12 inches inside the minimum rock-line. Thus it could be pushed past most projections and the tight rock marked for drilling and blasting. A second profile was fitted to the trailing end by means

of hinges, so that it could be collapsed out of the way. This was cut to the exact shape of the minimum rock-line and was used to check each section for clearance when trimming had been carried out.

CONCRETE MATERIALS

Difficulties in obtaining crusher plant and the necessity for using plant already at site imposed severe restrictions on the production of a well graded aggregate. In the earlier stages of the work, one crusher was used to produce aggregate of $1\frac{1}{2}$ inch maximum size, the products below $\frac{3}{8}$ inch only being separated since the range of the screening equipment was limited. A second crusher provided rock dust with a maximum size of $\frac{3}{8}$ inch, the use of rock dust for the fines being unavoidable owing to the very limited supplies of suitable river sand. Such river sand as was available was collected by a local subcontractor using somewhat primitive methods, but washing was carried out in a modern plant located on the river bank.

The mix produced with the equipment so arranged proved to be deficient in intermediate fines. Consequently, the separate production of dust was discontinued and the crushers were arranged to produce "crusher run" material, one set to limit the size to $1\frac{1}{2}$ inch and the other to 1 inch. A series of analyses of the products was made and, ultimately, by a suitable proportioning of these two grades of aggregate with a small quantity of river sand, a satisfactory and reasonably well graded mix was obtained which was used successfully throughout the remainder of the works.

Cement of Indian manufacture was supplied by rail to Bobbili, a station on the plains, north of Vizagapatam. It was then hauled by road to the site, a distance of about 130 miles, providing a major problem in road transport, for which the Electricity Department assumed responsibility.

PLANT AND EQUIPMENT FOR ARCH AND SIDEWALLS

Considerable thought was given to the method by which the concrete should be placed. By providing several shutter units, simultaneous hand placing can give a fair rate of progress. However, the time available called for a more rapid rate of construction than could be economically obtained by hand and it was believed that by the use of mechanical methods, the saving in labour costs would go a long way towards meeting the capital cost of the equipment.

Consequently, a 40-foot-long travelling steel shutter of conventional design was built in the United Kingdom and shipped out for service in both tunnels, additional parts being supplied for conversion from the pressure-tunnel section to that of the flume tunnel.

Of the two mechanical methods available, it was considered that a concrete-pump would be troublesome and unreliable in the hands of unskilled labour. This led to the decision to use a pneumatic placer.

Experience elsewhere had shown the placer to be mechanically reliable and free from the complications of blocked pipes which arise with inexpert handling of a concrete-pump.

The lining equipment comprised five units, all of which, other than the pipe tower, were linked together, so that they were drawn forward together, as required, by a hand winch set up ahead of the work (see *Figs. 5, 12, and 13*).

The first section consisted of a wagon ramp and belt elevator unit. The elevator was originally supplied with a special low-level boot or loading section to minimize the height of the ramp. Unfortunately, this section gave much trouble, the aggregate jamming under the steel guide plates at the sides, cutting into the belt. The boot was finally removed and a simple hopper built to discharge straight on to the sloping conveyor-belt, the wagon ramp being raised to the required higher level. It was arranged so that the wagons discharged on to a flat sloping portion at the end of the hopper rather than on to the belt itself through the hopper discharge opening. This called for one extra man who stood at the end of the hopper and, with a "pusher," fed the batch slowly and evenly through the discharge opening and on to the belt. This entirely eliminated jamming and reduced belt wear to normal proportions. With a few additional minor modifications, which included side rollers to prevent the conveyor belt wandering off centre, this unit gave no further trouble.

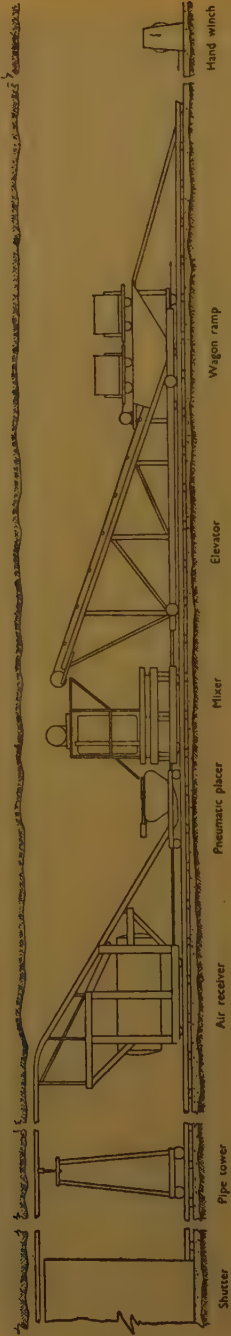
The belt elevator discharged into the storage hopper of a 1-cubic-yard mixer set up on a travelling chassis, comprising the second unit. On discharging the mix from the drum, the batch from the storage hopper was released, and the storage hopper again refilled by the conveyor. This gave a steady input, well up to the capacity of the mixer.

The discharge chute from the mixer led straight into the next unit, the pneumatic placer. The placer discharge-pipe was led up, and supported on, the carriage on which the air receiver was mounted, the last linked unit of the equipment.

Between the air-receiver carriage-support and the shutter, the pipe received further support from a pipe tower. The pipe tower travelled on the same tracks as the rest of the equipment, but was moved independently, as required, as the pipe was withdrawn from the shutter.

The batching plant (see *Fig. 14*), located outside the tunnel portal, was made at site, and consisted of timber hoppers fed both directly by wagons on an elevated track leading from the crushers, and by portable conveyor units fed from the crusher and washed-sand stockpiles. In front of each hopper, at a height which permitted the passage underneath of locomotives and wagons, there was a batching deck (see *Fig. 15*). Since the specification called for volumetric batching, fixed gauge-boxes were let into and flush with the deck, which simplified the process of screeding off surplus material. They were equipped with steel bottom doors fitted with counter-weights to make them self-closing, providing for their rapid discharge into

Fig. 5



EQUIPMENT FOR PLACING ARCH AND SIDEWALL CONCRETE

Fig. 10



DRILLING LOWER HOLES BENEATH THE PLATFORM

Fig. 11



THE WAGON SLIDE

Fig. 13



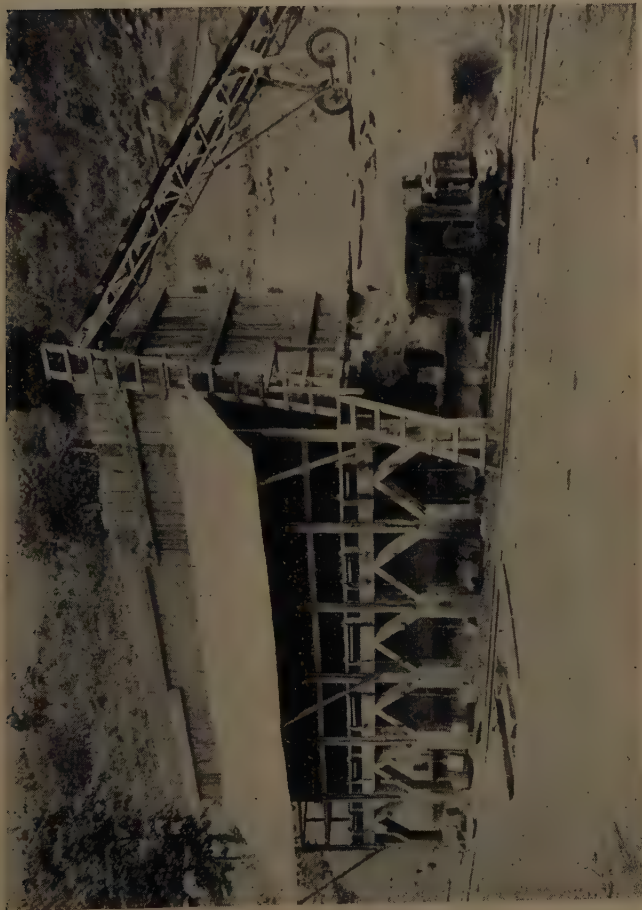
WAGON-MARSHALLING ARRANGEMENTS BEHIND THE TUNNEL-
LINING PLANT

Fig. 12



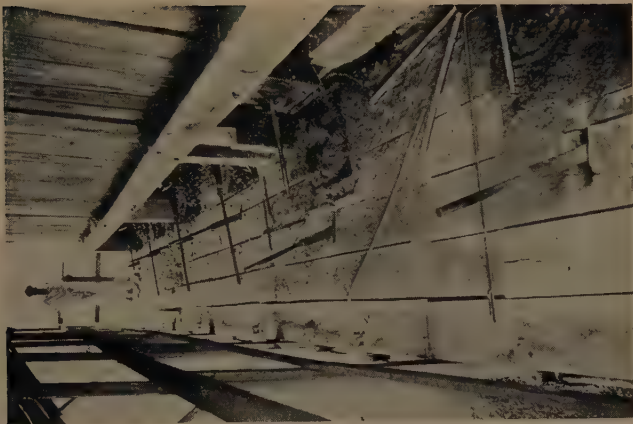
THE TUNNEL-LINING PLANT

Fig. 14



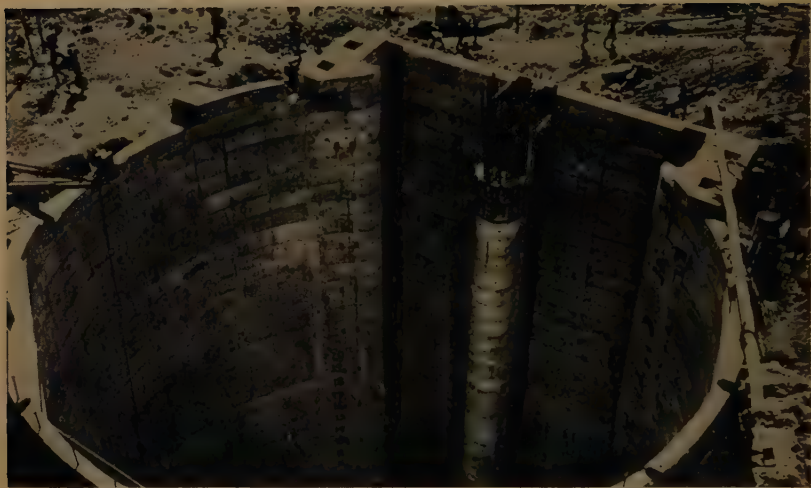
THE BATCHING PLANT

Fig. 15



CONTROL DECK OF BATCHING PLANT

Fig. 16



TOP OF COMPLETED SURGE SHAFT

Fig. 17



SHUTTERING ARRANGEMENT FOR SURGE-SHAFT WALLS

the wagons waiting below. The aggregates were fed to the gauge boxes through gate-controlled chutes. The sand hopper had simple gates which, with a little assistance from the operator, enabled the sand gauge-boxes to be filled rapidly without resort to shovelling.

The whole plant provided for two grades of aggregate and one of sand, the gauge boxes being duplicated in each section to permit simultaneous batching into two wagons at a time, at each stage throughout the train. Each wagon contained one mixer batch, and in this way, trains of batched material could be made up and dispatched at the rate required to meet the demands of the placer equipment.

Cement was added at the placer-plant elevator-hopper, just prior to use, the requisite number of bags being sent in on a flat bogie with each train.

PLACING THE ARCH AND SIDEWALLS

Work was started with key operators who had been trained on another smaller tunnel where similar equipment was employed, and with full knowledge of the pneumatic placer's tendency to produce segregation if not properly controlled. It was soon evident that the technique which had been used successfully on the smaller tunnel would have to be modified to obtain satisfactory results on the larger Machkund tunnels.

The smaller tunnel had a finished internal diameter of 9 feet. Using a fairly stiff mix, the bottom corners of the trailing end of the shutter filled quickly and, in a comparatively short time, the concrete built up on both sides to a steep slope and reached the crown. From this stage onwards, it was possible to keep the discharge end of the placer pipe buried to the extent of a foot or two, when the concrete flowed out in a manner similar to the discharge from a concrete-pump, and any tendency towards segregation of the materials disappeared.

In the Machkund tunnels, the vertical lift was nearly double that of the smaller tunnel and, presumably on account of this, it was found necessary to work with a rather wetter mix of 4 to 5 inches slump, if blocked pipes were to be avoided. This wetter mix gave a flatter slope to the concrete inside the shutter and this, coupled with the greater height of the arch, delayed the condition under which the buried-discharge-pipe technique could be employed, for several hours after the start of the pour. In fact, this condition was not reached until the sides were filled to approximately the springing level of the arch. Also, during this period, the flow of concrete to each side was not always even and a careful watch had to be kept on this, the discharge pipe being moved over to one side or the other, as necessary, to avoid eccentric loading of the shutter.

On striking the shutter, in the early stages of the work, serious honey-combing was found at the bottom corners at the trailing end. The remedy adopted was to shoot about six batches of sand/cement mortar first, before shooting the normal concrete. This completely eliminated honey-combing in this area.

During the next phase of the operation, until the springing level of the arch was reached, satisfactory results were obtained only by a combination of vibrating and hand punning through the inspection panels provided. The original external-type vibrators obtained were too light, but successful results were obtained with heavy insertion-type vibrators used externally by mounting them in suitable clamps fitted to the main ribs of the shutter. The hand punning was closely supervised and the best results were obtained by running the vibrators for 3 or 4 seconds only, after the hand punning had been carried out and just before the next shot from the placer.

Once the condition was reached in which the discharge pipe could be buried and progressively drawn out as the arch filled up, little trouble was experienced other than what might be termed a dirty-shutter effect. During the pouring of the walls, a certain amount of concrete splashed over the crown of the shutter forming a patchy skin, less than 1 inch thick which, of course, set. If this was left in this condition, the splashed material tended to cling to the shutter on striking, so leaving an uneven finish to the crown. Thus, it was necessary to stop work for a short period when the sides were filled and, wherever the roof clearance permitted, to clean the shutter crown before proceeding to fill it. The buried-pipe technique and the compacting effect of the placer discharge inside the limited space of the arch produced sound concrete, all normal roof cavities being filled. Honeycombing never appeared in the arch.

Interruptions to pouring from any cause tended to produce "cold joints"—a layer of honeycombed concrete lying immediately above the concrete surface formed when the stoppage occurred. This trouble was largely eliminated by first chipping the surface whenever it was accessible and, on resuming the pour, shooting in first a batch or two of sand/cement mortar before following on with the normal concrete mix.

One slight surface defect appeared in the form of tiny holes resulting from small bubbles of air adhering to the surface of the shutter on the sidewalls, never in the arch. This trouble tended to increase whenever the shutter was over-oiled and in spite of all efforts, was never entirely eliminated.

A further surface defect in the form of water marks proved to be very difficult to avoid in the pressure tunnel, which had sloping sidewalls. The necessity for using a rather wet mix resulted in the appearance of small quantities of water on the surface of the wet concrete after a height of 4 or 5 feet had been reached in the sidewalls, continuing up to the springing level of the arch.

It was considered that this surplus water was squeezed out of the lower levels of concrete by the weight of concrete added above, and that the water travelled *upwards* to the surface. In the pressure tunnel, where the walls were inclined at an angle to the vertical, water, which was squeezed out of the concrete near to the face of the wall, rose vertically and

tended to strike the surface of the steel shutter. It continued to travel upwards on the surface, taking with it minute quantities of the surface cement grout.

Striking the shutter, at these levels, revealed a series of "rivulets" giving a sandy surface of the order of $\frac{1}{16}$ inch thick. This sandy surface could be brushed off showing perfectly sound concrete underneath. Thus, it had no deleterious effect of the strength of the lining but it spoilt the general appearance of the finished work.

The theory that the inclined sidewall shutters of the pressure tunnel were largely responsible for this surface defect was supported by the fact that it appeared in very few places in the flume tunnel, where the sidewalls had a vertical face.

A minimum setting-time of 48 hours was at first imposed in the initial stages of the work by the Government engineers. Naturally, the contractors wished to improve upon this to gain the maximum advantage from employing mechanical methods of placing the lining. Both parties co-operated in tests for arch settlement, strength, and other factors, and in due course, as a result of these tests, a setting-time of 16 hours was authorized in good ground where no rock load would be imposed upon the lining.

With this concession, and once the proper technique was mastered, progress reached the average of five lengths, or 200 feet of lining, per week.

The general conclusion to be drawn from the Authors' experience in the use of the pneumatic placer is that, with due regard to the proper control of the mix, water content, punning, vibration, and so forth, the machine is entirely satisfactory for placing the whole arch and sidewall lining in one operation in small tunnels of about 10 feet diameter. Difficulties in controlling the flow, and segregation problems increase rapidly with larger tunnel diameters, and tunnels about 15 feet in diameter appear to approach the limit of successful operation. Even in this size, and certainly in excess of this, it would seem advisable to place the sidewalls with one unit, employing hoists and chutes, and to place the arch with another unit following behind at a suitable distance, employing the pneumatic placer to deal efficiently with this portion of the tunnel lining where other methods are both costly and often unsatisfactory.

USE AND MAINTENANCE OF PLACER EQUIPMENT

The efficient operation of the placer equipment depended upon comparatively small details, but any of these could give trouble which, if it occurred when concreting, led to unnecessary delays and the risk of "cold joints."

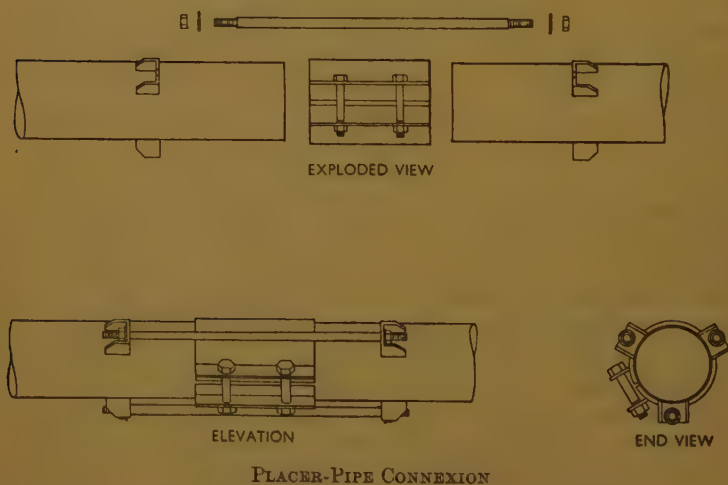
Wear in the placer itself was not abnormal and build-up welding of the throat was required only on completion of the first tunnel, when about 8,000 cubic yards of concrete had passed through and, of course, on

completion of the second tunnel. The placer was free from mechanical defects throughout its use.

The top bend of the discharge pipe was of special design and provided with replaceable wear plates. These were renewed every few lengths and their order changed between pours to ensure equal wear. In addition, parts of the housing and the bend itself required building up at fairly frequent intervals. By working with two units, one in use and one under repair, no time was lost on this account.

Six-inch steel pipe was used for the delivery and it soon became evident that screwed joints, or any form of joints which reduced the metal thickness, were unsuitable. With such joints, pipe life was limited by the thickness of the metal at the joints, and failure occurred long before the pipe itself wore out. A simple butt joint, which was devised at site, and which left the metal thickness unimpaired, is shown in *Figs 6*.

Figs 6



Pipe wear was greatest along the bottom of the pipes, and to counteract this the pipes were rotated through 120 degrees every length. The pipes and joints thus wore uniformly giving the maximum available life.

Wear was least near the bend and greatest near the discharge end where the concrete attained its maximum velocity. Thus, the end pipes had a shorter life than the other pipes in the set but, by using a progressive system of replacement, it was possible to keep the consumption of pipe to the minimum. On average, a set of pipes sufficed to complete about twenty lengths.

Although the above-mentioned points were well known to all, details were apt to be overlooked resulting in unnecessary pipe wear and delays

which had an adverse effect upon the quality of the work. The problem was overcome by the production of a form * which was given to the foreman by the engineer after the latter had checked the setting of the shutter. The form listed each item of the routine to be followed by the concrete gang in the maintenance and preparation of the concreting equipment prior to commencing a pour. The foreman was required to check through and sign every item on the form before giving the order to start. Whilst a signature was no absolute guarantee that the work had been carried out, the form served as a reminder of each step in the routine and pinned down the final responsibility to the foreman with the result that delays arising from any failure to comply with the routine seldom occurred once this procedure was established.

FINAL INVERT CONCRETE

The tunnel linings were completed by placing the final dished invert over the initial floor.

The sidewall shutter was designed to give a projecting strip supported by the kerb along the foot of each wall, the top of which marked the intersection with the finished invert. In practice, this was not entirely successful. In pouring the sidewalls, the concrete in this bottom strip set long before the arch was completed. No matter what precautions were taken to bed down the shutter tracks securely, a slight vertical settlement took place as the load of concrete built up over the arch. Thus, on striking the shutter, it was invariably found that the projecting strip had sheared off along most of its length. However, some parts survived, and it was made up elsewhere by hand to such an extent that it could be utilized as a screed.

The Authors believe that the alternative practice of casting-in a "corner piece" along each side of the tunnel first, using a steel shutter, is to be preferred. The work entailed in setting a shutter accurately is no more than that which was required for the kerbs used in this design, and such precast "corner pieces" are unaffected by any slight movement of the wall and arch shutter which follows.

The final invert was poured in two halves, the first half following close behind the main shutter and poured during the setting period. The second half was not concreted at this stage since it served as a drain, the lining being placed in an uphill direction. A screed was fixed along the centre of the tunnel floor and concrete from the main mixer was run in wagons through the shutter on a special track which by-passed the placer unit. The concrete was placed by hand and trimmed to profile by working a moulding screed between the wall projection and the centre screed.

As the work proceeded, the 2-foot-gauge track ahead of the plant was

* The wording of this form is given in an Appendix (unpublished) to the MS. of this Paper.—SEC. I.C.E.

picked up, carried through the shutter and re-laid behind on top of the finished half of the invert. The track was therefore ready in position when the arch and sidewalls were completed for running in the concrete required to complete the second half of the invert. This was, of course, laid downgrade so that all accumulated water was led away ahead of the work and drained out at the tunnel exit. The correct dished shape was formed by screeding between the wall projection and the surface of the concrete already laid in the first half.

SURVEYS AND SETTING-OUT

Independent triangulation surveys were made for each of the two tunnels with the primary object of determining their lengths. Open traverses were also made as a check on the alignment. These surveys presented no particular difficulties other than those imposed by climatic conditions which limited instrument work to short periods near sunrise and sunset.

Since few members of the tunnel crews could read, and generally to eliminate the possibility of errors through the misreading of tapes or rules, all setting-out was planned for the use of sights and gauges.

Whilst driving the tunnels, the line was carried on roof stations marked on plugs driven into holes drilled in the roof at approximately 80-foot intervals. Candles were set to line and axis level in holders slung from chains attached to the roof stations. These were used by the tunnel foremen and senior shift bosses to sight and mark the axis point of the face. From this, the positions of the holes were marked out with gauge sticks for drilling.

The chainage was marked up at 50-foot intervals, and at each of these points level-pegs were driven into holes on both sidewalls at a fixed distance above invert level. By stretching wires across two sets of these level-points, the tunnel foremen were able to bone ahead and keep close control over the floor grade and track levels.

The same roof stations and side level-pegs were used for trimming the floor and for boning-in the initial invert screeds.

Since the kerb edges had to be set accurately to line, short pieces of rail were cast into the centre of, and flush with, the surface of the initial invert. On these, an engineer marked the tunnel centre-line, and the kerb shutters were set from the marks by means of a steel gauge.

The trimming-gantry tracks were laid on the initial invert with the sleeper ends just touching the kerbs, the spacing of the bogies having been designed with this object in view. The gantry and, consequently, the trimming profiles were thus automatically set to line and level throughout the tunnel. The setting was sufficiently accurate for taking the final cross-sections. These were plotted from offsets taken at 2-foot intervals round the fixed profile after trimming had been completed.

For setting the tracks for the shutter, a steel gauge was provided

which rested on and between the kerbs, thus enabling the tracks to be laid accurately to line and within the required limits of level.

Ahead of the shutter, pins were grouted into each kerb at the exact chainages which would correspond to the leading edge of the shutter allowing for the necessary overlap with the previously concreted length at the trailing end. The pins were also set to a fixed grade so that they projected about 2 inches above the kerbs.

After the shutter had been drawn forward and correctly positioned by bringing the leading edge into line with the pins, a straight-edge was laid across the pins and, using a vertical gauge stick, the crown was jacked up to level. The track-setting gauge was then used to check the alignment at the crown by dropping a plummet from the centre marked on the crown of the shutter at its leading edge. All these operations were performed by the concrete foreman, but the final settings were usually checked by an engineer as an insurance against mistakes.

For the final invert, since grade pegs set in the floor ahead of the lining equipment were liable to be damaged, new boning pins were grouted-in by an engineer for setting the centre screed.

SURGE SHAFT AND PIPE TUNNEL

The contract for the tunnels also included the construction of a concrete-lined surge shaft (see *Fig. 16*), 50 feet in diameter and about 75 feet high, and a concrete-lined pipe tunnel leading out of the surge shaft, 32 feet wide by 21 feet high, with a length of about 110 feet.

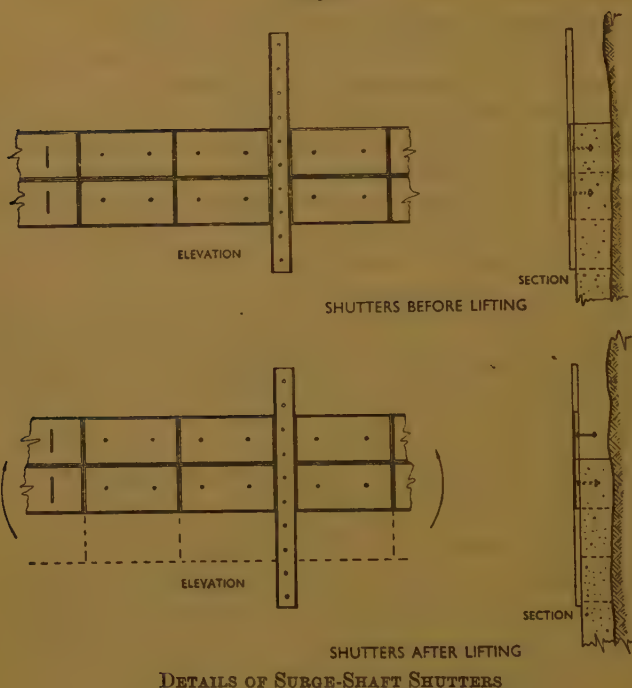
On the surge-shaft site, a small pilot shaft was first sunk from a preliminary open cut on top. On completion of the pressure-tunnel drive, the shaft was opened up to full size, using the pilot shaft as a muck chute and loading by rocker-shovel below.

Owing to protracted negotiations with the manufacturers of various special equipment, the final drawings for the surge-shaft concrete were available only after the works were well advanced. It was not possible, therefore, to have shuttering manufactured in the United Kingdom and shipped out in time, and its fabrication had to be carried out in the contractor's site workshops with such materials as were available.

A self-supporting system of steel shuttering was designed in the form of two sets of rings, each 2 feet high, divided into panels with channel-section soldiers, 10 feet long, located at suitable intervals to help in retaining the plumb of the walls (see *Figs 7 and 17*). Since no suitable angles were to be obtained, the panels were built up from $\frac{1}{4}$ -inch plate and framed with 2-inch-by- $\frac{1}{4}$ -inch strip, fillet-welded all round. The task was accomplished by the use of special jigs which effectively prevented distortion of the panels during the process of welding.

Sockets consisting of nuts welded to 2-inch-square washers were cast in the concrete to receive fixing bolts as work proceeded. The soldiers were bolted into the walls by means of these sockets, as was the lower

Figs 7



ring of panels. New sockets were cast in while pouring the upper ring. When the concrete had set, the bolts were checked for tightness in the upper ring and the lower set of shutters was released. These were then erected on top of the previous upper ring, all the panels being bolted together and to the soldiers, and the process repeated. The soldiers were lifted every third pour, and had always a lap of at least 4 feet over the finished work to maintain rigidity and plumb.

An independent system of timber scaffolding was constructed to provide circular platforms at vertical intervals of 6 feet. The concrete materials were brought through the pressure tunnel to a mixer located at the bottom of the surge shaft. This mixer discharged into a site-made bucket which was lifted to the required platform level by means of an air winch. From the platforms, the concrete was shovelled into the forms by hand. Progress under dry-weather conditions averaged about two and a half lifts, or 5 feet, per week.

The pipe tunnel was enlarged to full size from the heading which originally provided access to the pressure tunnel. A preliminary concrete floor was first laid and the sidewalls were cast-in to springing level of the arch.

Since the tunnel arch was of considerable size, it was considered that

much time and labour would be saved if the shutter was constructed as a one-piece travelling unit. A length of 7 feet 6 inches was selected as being the most suitable. Since any form of jacking for setting-up and striking called for vertical movements, and since the tunnel was on an appreciable grade, the ribs were formed on a vertical plane and were therefore of a slightly elliptical section. The ribs and runners were made up from timber with a surface lagging of 12-gauge sheet-steel plates.

This unit was bolted to a cross-frame made up from 5-inch-by-4½-inch R.S.Js and this was supported by four sets of 6-inch-pipe columns. The bottom ends of each set were welded to a 10-inch-by-3-inch channel forming a soleplate. Under each soleplate, an 8-inch-by-6-inch R.S.J. was bedded to grade on the initial concrete floor of the tunnel, leaving a gap for the insertion of timber wedges to maintain the shutter at its correct level (see *Figs 8*).

In the two gaps between each outside pair of supports, 2-foot-gauge tracks were laid. On each of these, a frame was built up on two bogies with a capping piece to give a few inches clearance under the cross-framing of the upper part of the shutter. Thus, by the use of ordinary jacks at floor level, and by knocking out the wedges beneath the soleplates, the whole shutter could be lowered on to the two carriages, leaving the column supports hanging free. By means of a hand winch, the whole double unit was then drawn forward to the next position, and the shutter jacked up off the carriages and wedged in position, ready for the next pour.

After setting up the shutter, the carriages were drawn forward and decking was laid across the top to form a working platform for the men. Concrete from the mixer was discharged directly on to the belt of a portable elevator which, in turn, discharged the concrete on to the carriage platform. From there, it was manhandled into the shutter (see *Fig. 9*).

With a setting time of 24 hours, it was possible to concrete about three lengths, or 22 feet 6 inches, per week with this equipment.

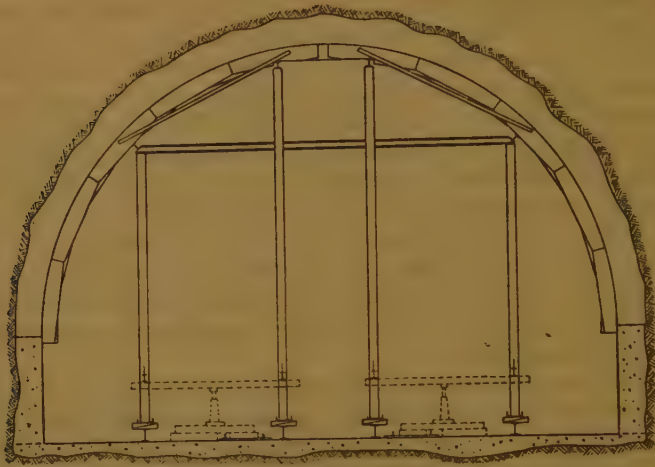
CONCLUSION

Although the contractors had made their decision, considerable doubts existed at first as to whether or not it would prove to be practicable to combine the employment of modern plant and methods with untrained labour under the conditions which obtained at Machkund. These early doubts soon proved to be unfounded.

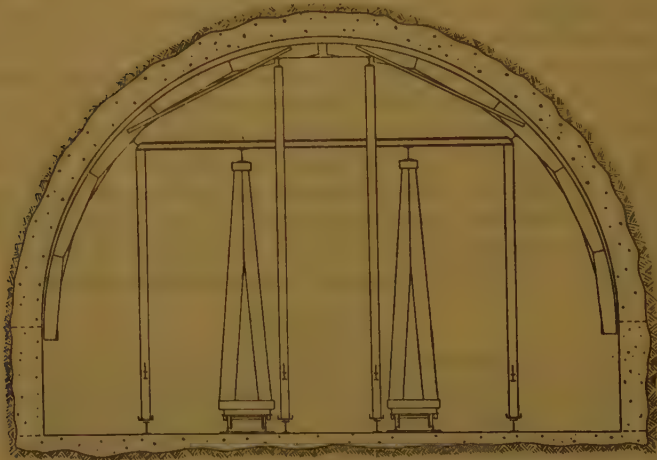
By suitable training and supervisory control, the men acquired a remarkable degree of skill. The mechanical equipment served well, and the men soon dropped into the routine operations of maintenance, driving the tunnels, and placing the linings. Progress bonus was introduced throughout as many operations as possible, and this kept interest and effort at the consistently high level necessary to maintain good progress.

To sum up, the correctness of the contractor's decision was proved by the results obtained which compared very favourably with western standards.

Figs 8



SHUTTER SET IN POSITION



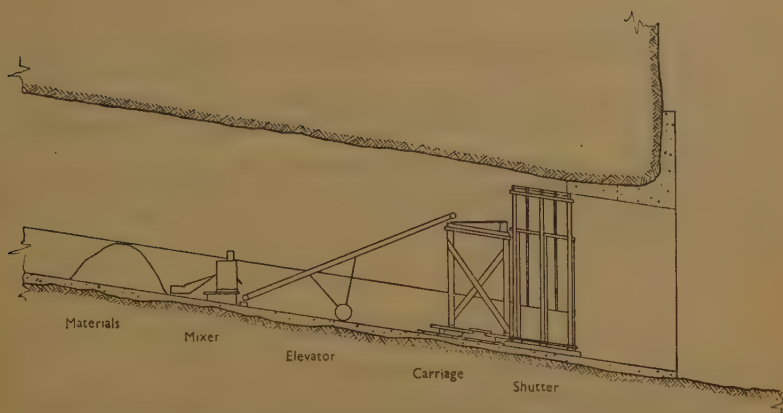
SHUTTER LOWERED ON TO CARRIAGE FOR MOVING

PIPE-TUNNEL ARCH SHUTTER
(STRUTTING DETAILS OMITTED)

ACKNOWLEDGEMENTS

The Authors wish to express their thanks to Mr S. R. Krishnamurthy, B.E., Consulting Engineer (Civil) to the Electricity Department, Government of Madras, Mr P. M. Chengappa, B.E., Superintending Engineer at

Fig. 9



ARRANGEMENT OF EQUIPMENT FOR CONCRETING PIPE-TUNNEL ARCH

the site during the major portion of the construction of the tunnels, and to Sir George Burt, M.I.C.E., for their kind permission to produce this Paper and to make use of various drawings from which some of the illustrations have been prepared.

The Contractors were Messrs John Mowlem & Co. Ltd.

The Paper is accompanied by eight photographs and nine sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared, and by four Appendices, three of which are reproduced in the following pages.

APPENDIX I

ROCK TUNNELLING CYCLE-TIMES

The cycle-times quoted below are for the flume tunnel and exclude major delays arising from factors beyond site control, such as electrical supply failures, etc., but include minor delays such as derailments, plant breakdowns, etc.

Since the pull per round was not constant throughout the driving of the tunnel owing to the different lengths of drill steels used, a typical average cycle-time is given for an 8-foot pull, involving drilling 65 holes and loading 150 one-cubic-yard wagons of muck.

	hrs	mins
(1) Waiting for fumes to clear after the blast before re-entering the tunnel	0	50
(2) Taking equipment up to face, barring down loose rock, clearing track, reconnecting manifolds, etc., in preparation for mucking	0	30
(3) Mucking, including an average of 30 minutes delay whilst erecting platform for drillers	8	00
(4) Drilling lower holes	3	00
(5) Charging face, dismantling platform, removing equipment from face, testing circuit, and blasting	1	45
Total	14	05

Notes

(a) While mucking was proceeding, the face was marked and all holes drilled above the lower holes, which latter comprised only 40 to 45 per cent of the total number of holes.

(b) For rounds giving pulls of up to 9 feet 6 inches, the average times for items 1, 2, and 5 were found to remain substantially the same and the times for items 3 and 4 only were increased.

(c) The total cycle-time was largely controlled by the mucking time (see text) and could this have been reduced, the total average time could easily have been reduced, for the drilling of the middle and top holes was invariably finished long before mucking was completed.

APPENDIX II**PERFORMANCE OF DRILL STEEL AND BITS**

The figures quoted below are for the flume tunnel only, where both detachable bits with tungsten-carbide inserts and tipped steels were used.

Detachable Bits and Drill Steel

These were $1\frac{1}{2}$ inch diameter, of cruciform shape, with a female tapered socket which was a push-on fit using a copper sleeve to the corresponding male taper formed on the end of the drill rod. The drill rods were 1 inch hollow hexagonal and both shanks and tapers were fashioned on site.

Total lineal footage drilled	209,511
Total number of copper sleeves expended	4,300
Total lineal footage of drill steel expended	3,950
Number of long drill steels fashioned or refashioned on site	3,755
Number of short drill steels fashioned or refashioned on site	2,435
Total number of shanks broken during drilling	2,188
Total number of tapers broken during drilling	906
Number of detachable bits that became unserviceable	1,133
Number of detachable bits lost during drilling	398
Total number of detachable bits expended	1,531
Average lineal footage drilled per detachable bit	136.8

Tipped Steels

These were chisel pattern giving a $1\frac{1}{2}$ -inch-diameter hole with tungsten-carbide inserts, and were supplied shanked and ready for use by the manufacturers.

Total lineal footage drilled	68,727
Number of tipped steels rendered unserviceable by damaged or worn-out carbide inserts	203
do. by broken shanks	19
do. by broken tapers	4
do. by jamming in holes so that they could not be extracted	13
Total number of steels consumed	239
Average lineal footage drilled per tipped steel	287.6

Notes

(1) If the tipped steels with broken shanks had been reshanked on site and put into use again, it is estimated that their remaining life was equivalent to 7.20 new steels, and this would have increased the average footage per drill steel to 296.5 lineal feet or only 3.1 per cent.

(2) The consumption of long and short steels was not equal and it was found that 139 long steels were used as against only 100 short steels, owing to the practice of drilling the lower holes with a long steel from start to finish.

Comparative Costs

The tipped steels gave a much better footage per steel than the detachable bits but their initial cost was much higher. On the other hand they required no maintenance at site other than the grinding of the carbide tips, which was necessary with the detachable bits as well.

If the cost of grinding, which is both small and common to both, is ignored, the comparative cost based on the initial price of a detachable bit being taken as 100 units were as follows :—

	<i>Cost per 1,000 ft of drilling</i>	
	Detachables Bits	Tipped Steels
Initial cost of bits or steels used	732 units	690 units
Cost of hollow steel used	64 „	nil
Shanking and tapering drill steel, labour, and materials	71½ „	nil
Cost of copper sleeves used	20 „	nil
Total	887½ units	690 units

Whilst this shows that the costs for the tipped steels were less, the poor quality of the hollow drill steel available, which was used with the detachable bits (as noted in the text), was responsible for the heavy bit losses and the cost of shanking and tapering. If the quality of the drill steel had been such as to reduce the losses of bits by two-thirds and the cost of the drill steel consumed (plus the cost of fashioning it) by a half, there would have been little difference in costs.

APPENDIX III

CYCLE-TIME IN BAD GROUND SECTIONS

The cycle-times quoted are for an 8-foot length in the flume tunnel and represent the average times taken once the routine was established and the first two or three sections completed. The initial sections took considerably longer for the gangs had had no prior experience of this work.

	hrs	mins
(1) Waiting for fumes to clear after the blast before re-entering the tunnel	0	50
(2) Taking equipment up to face, barring down loose rock, cleaning tracks, reconnecting manifolds, etc., erecting temporary roof support	3	0
(3) Mucking	7	30
(4) Trimming the sidewalls and arch where necessary	2	0
(5) Cleaning and trimming the invert at the sidewalls	3	0
(6) Bringing in and erecting the shutter frame and the first few feet of laggings	6	0
(7) Concreting, including fixing rail reinforcement and the remainder of the laggings and dismantling the temporary roof support where necessary as work proceeded	18	0
(8) Setting-time for the concrete, during which some holes for the next round were drilled	12	0
(9) Dismantling the shutter	4	0
(10) Completion of drilling	2	0
(11) Charging the face, removing equipment, testing circuit, and blasting	1	15
Total	59	35

Discussion

The President, when proposing the vote of thanks to the Authors, said he had gained the impression, when reading, that the work had been carried out with an unusually small number of Europeans. The Authors must have trained their Indian subordinates extremely well to have been able to give them so much responsible work to do. The site had clearly been extremely difficult of access, necessitating much improvization. No attempt had been made to get a highly finished temporary works but everything had been done in a way very suitable for the work.

Mr A. P. Lambert observed that the main value of the Paper lay in its straightforward and honest description of the work which the Authors had carried out. It was particularly valuable because the Authors had described some of their problems, how they had tried to solve those problems, and how in some instances they would do things a little differently another time; for instance, the ventilating ducts had not been quite large enough and the fog at the tunnel face might have been reduced if there had been after-coolers and separators to get rid of the moisture before the air went into the tunnel. The relationship of the size of the wagon to the rocker-shovel had also been mentioned. A Paper in which the Authors frankly stated their difficulties and confessed that they had not quite overcome them was very much more valuable than a Paper where there seemed to have been a magician in the background, where no problems ever occurred, and where any difficulties were solved before anybody knew anything about them.

He thought that the Paper was lacking in information in one or two places, and it would be of interest if those gaps could be filled in. He could not find any statement about the thickness of lining which had been asked for. There was given in *Figs 1 (b)* a cross-section of the tunnel, and the dotted line inside the face of the tunnel looked as though it might be 6 inches from the face and that might be the line in question. The information, if it could be given, would make it possible to compare the thickness of the lining in this case with the thickness used elsewhere. It was also a matter of great importance to the contractor, because the influence of the concrete in the over-break was affected by the thickness of lining; if the contractor had already to provide a thick lining, the concrete which he lost in the over-break was relatively less important, and he might take a slightly different line when blowing his tunnel; it might pay him to insure against the possibility of having to undertake expensive trimming by cutting the rock a little wide. On p. 136 reference was made to the final cross-section of the tunnel; what was the average thickness provided compared with the minimum thickness asked for?

Referring to the statement on p. 128 that "Consequently, a 40-foot-long

travelling steel shutter " had been sent out from the United Kingdom, Mr Lambert asked the reason for the word " consequently." Why had a length of 40 feet been chosen ? He believed that a contractor planning the lining of a tunnel would find that the calculation of the economic length of shutter was very interesting, and also extremely important to him so far as the economic results of his contract were concerned. It was probably almost the most important decision which the contractor had to make. If one shutter were twice as long as another, it could be agreed that it would cost nearly twice as much ; but, having accepted that, there seemed to be many benefits which devolved from the longer shutter. For instance, the stop ends, which were troublesome in any case, would occur only half as often. If the tunnel was to be lined with a pneumatic placer it was the common experience that difficulties with that technique, if they occurred at all, occurred at the start of the pour ; once the concrete had reached the crown of the arch and the end of the discharge pipe could be buried, everything went much more simply. If the shutter were twice as long that troublesome period occurred only half as often.

In the case described, the " waiting time " had been 16 hours after filling the shutter, and the Authors had obtained an average of 5 moves per week. That meant a total of 80 hours, and there were only 168 hours in a week, including the week-end, which probably had not been worked, so that rather more than half the effective working time had been occupied in waiting for the concrete to go off. It would still be 16 hours with a shutter twice as long, so that the effective lost time would be halved. Mr Lambert did not want to suggest that in the case in question the shutter should have been twice as long, but perhaps the Authors would say a little more about their calculations, and their reasons for deciding on a length of 40 feet, which on the face of it seemed to be rather short.

Would the Authors state the cement content of the mix, so that it would be possible to compare the richness of that concrete with concrete in other tunnels. Mr Lambert thought that those in charge of the work on the site had been under a very great disadvantage with the concrete which they had had to use, in the first place because only a very small proportion of natural sand had been available, and crusher-produced sand was a very unsympathetic material ; with crusher-run aggregate, particularly of hard igneous rock, the aggregate was very liable to segregation in the handling. Those factors meant that it was desirable to have better-than-average facilities to split the aggregate into sizes and re-blend it correctly for the concreting, but the Authors had had considerably less than the average facilities for doing that. Those who had used concrete in any situation which was a little unusual, and where they were trying to follow a rather special technique, would have found that there was a mix and a grading, which would suit their requirements, and, having found that grading it was of the utmost importance to be able to preserve it. He did not believe that the Authors had had the necessary facilities

to preserve it, and a good many of their difficulties in the concreting might have devolved from that lack of adequate control.

That led Mr Lambert to question the conclusion on p. 133, that the method of using the pneumatic placer was unsuitable for tunnels more than 15 feet diameter. That might be so, but he thought that that conclusion must be accepted with a reservation until the equipment could be tried out under conditions more favourable for concreting.

An important aspect of lining a tunnel with the pneumatic placer was the actual rate of concreting. Whatever the size of the tunnel, the concrete had to be put in at such a pace that the wet end of the concrete was kept moving forward at a satisfactory rate, and that satisfactory rate must be such that the concrete—any concrete—was completely submerged and compacted and buried by other concrete before it started to go off. It followed from this that, as the size of the tunnel increased, so the rate of placing must increase in order to preserve that uniform movement along the tunnel, and it might be that with a 15-foot tunnel the rate of placing by the one placer was not adequate.

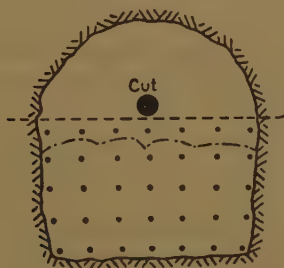
Could the Authors give further data about the rate of placing? Mr Lambert had calculated from the figures given in the Paper that each pour had been between 105 and 110 yards of concrete, but the Authors had not stated whether it was a 3-hour or a 6-hour job; he felt that with the placer equipment which had been used, the quicker the rate of emplacement, the better would be the concrete. A state of affairs might be arrived at where, in a very big tunnel, it was desirable to double the equipment, but to arrive at the conclusion that it was necessary to use one type of apparatus to do the crown and a different sort of apparatus to do the walls was one which he did not think could be accepted without exploring the alternatives more fully.

Mr W. B. Tozer remarked upon the fundamental advantage of overlapped drilling and mucking. In a normal cycle, in which drilling, blasting, and mucking succeeded each other, the drilling time was regulated not by the maximum number of machines which could be crowded into the face but by the minimum number of holes which one machine must deal with, and the cut-hole pattern usually decided that, particularly if it was a burn cut, such as was used in the tunnel in question.

In the case described, he thought that six cut holes had been shown in the film with which the Authors had introduced the Paper, and in addition the driller putting in those six cut holes would probably also have to put in the easer holes immediately above and below, which meant eight. The drilling time, therefore, if a normal cycle was used, would depend on how long it took to put in those eight holes (the six cut holes and the two easers) or possibly more. It would be useless to reduce the task of the other drills to less than eight holes. It was stated in the Paper that only 40-45 per cent of the holes had remained to be drilled after the mucking had been completed. The chain-dotted line in *Fig. 18* was

supposed to represent the top of the muck pile, and it was only the holes below that which had to be drilled, and which of course added to the length of the cycle, the other holes being drilled while mucking was going on. The Authors had also stated that a total of sixty-five holes had been drilled, which left twenty-eight to be drilled with seven machines. That reduced the total task per machine to four holes or sometimes five in a vertical row. In other words, what Mr Tozer suggested was that the potential drilling time had probably been halved in the present case by using the overlapping system compared with the normal cycle.

Fig. 18



He noticed that the Authors had made some criticism of the design of the platform and he could imagine that it had given a certain amount of trouble in bad ground. Some experimental work had been done recently in an endeavour to overcome the more obvious snags in that respect. A much simplified platform structure, which was lighter, cheaper, and much easier to erect, had now been evolved. Tests had been made on a model which spanned an 18-foot tunnel and required only one wall hole at the end of each beam. It would comfortably carry a live load at the centre of at least $\frac{1}{2}$ -ton, which corresponded to about five drillers and machines. The beam was simply a piece of pipe about 4 inches in internal diameter, with a $1\frac{1}{2}$ -inch sprag in high-tensile steel and a split conical clamp which clamped the sprag when inserted in the hole and centred the pipe on the sprag. It allowed considerable misalignment of the support holes and therefore overcame many of the difficulties. Another advantage was that pieces of pipe were readily obtainable, and in the event of overbreakage it was a fairly simple matter to provide a few slightly longer pieces of extra pipe somewhere near the face and then pick out the size required.

Mr William Brown said that as a contractor he was very interested in the rate of progress, but he could find no reference in the text to the sizes of the tunnels, by means of which he would make comparisons. It would appear from *Figs 1 (b)* that the tunnels were of about 16 feet equivalent diameter, whilst the maximum progress was given as 130 feet per

week. In Appendix I an average time-cycle of about 14 hours was given and, assuming the usual 136-hour working week, that showed a weekly progress of about 80 feet, which he thought was very creditable. In the report of the North of Scotland Hydro-Electric Board it was stated that their rate of progress in 16-foot tunnels driven in Scotland had improved from about 40 feet per week in 1949 to a little more than 90 feet per week in 1952. The Board had also stated that in their 10-foot tunnels the rate of progress had gone up from about 70 feet per week in 1949 to about 135 feet per week in 1952. The majority of those tunnels had all been driven by the modern technique described in the Paper, with light-weight drills mounted on air-legs and "Eimco"-type loaders filling 2-yard skips on 2-foot-gauge track, hauled out by Diesel or battery locomotives.

It was generally agreed that 2-yard skips on 2-foot-gauge track represented an economical proposition for tunnels of the size in question and it was no surprise, therefore, to read the Authors' conclusion that their 1-yard skips had been too light.

Mr Brown had been surprised to read that the Authors had had no trouble from exhaust fumes with Diesel locomotives, particularly since the methods of dealing with ventilation had not been all that they might have been. Even though poisonous fumes might be avoided, there was still the characteristic Diesel smell, and in the United Kingdom there was a temptation on that account for labour to invoke the clause dealing with obnoxious conditions. Mr Brown would advocate where possible using battery locomotives in a tunnel.

Referring to the concrete lining, he had been interested in the Authors' decision to use a placer for easier handling by native labour and because it was less likely to choke the pipes than a concrete-pump. He agreed that there was nothing more frustrating and aggravating than a choked pipe-line, particularly where a length of 400 or 500 feet was concerned, but he still thought that the concrete-pump had much to be said for it as a means of placing concrete for tunnel lining. The merits of the placer, he understood, lay largely in the rate at which it was possible to shoot the concrete into a shutter, but he thought that undoubtedly a certain amount of segregation occurred and that there was not very good control of compaction and the consistency of the concrete. Vibration might also be a difficulty. He understood that with a concrete placer it was not possible to have men using internal vibrators when concrete was being shot in at an air pressure of 100 lb. per square inch, and the Authors had resorted to external vibrators fitted to the ribs. In the United Kingdom some engineers frowned on that, and Mr Brown thought that external vibration was very local in its effect. He knew that comparison between concrete-pumps and concrete-placers was a question of some delicacy, but he personally believed that the concrete-pump was a very much better tool for pressing concrete into the roof. Especially in a steel-lined tunnel with steel ribs and packing and plenty of voids it was possible to get a pressure

on a concrete-pump and a consistency in pumping which he did not think was practicable with a placer.

The Authors had concluded that difficulties in controlling the flow and the problem of segregation increased rapidly with larger tunnel-diameters. They believed that a diameter of about 15 feet was approaching the limit at which a placer could be successfully used for placing the whole arch and sidewall lining in one operation. Mr Brown thought that that was probably correct with a placer, although he admitted that he had not used one. Recently, however, in the Woodhead tunnels, a combined arch and sidewall shutter had been successfully used in a tunnel of about 28 feet finished diameter; the concrete had been placed by means of four pumps, and the output had reached the maximum of about 3,000 cubic yards per week, the average over many months being about 2,000 cubic yards.

He suggested that just as in tunnel excavation the rate of driving depended very largely on the rate at which it was possible to get rid of the muck, so in tunnel lining probably the speed of lining would depend mainly on the rate at which it was possible to supply aggregates, sand and cement to the tunnel.

Brigadier S. A. Westrop explained that he was in a rather fortunate position compared with previous speakers, because he knew the works described intimately and had examined them during the course of progress at various stages.

Certain items of plant, such as the light capacity tip trucks, had been criticized, and he wished to explain that the contract had been of the target type; and the Madras Government had already supplied a good deal of plant for the job, including tip trucks and crushers, which the contractors had had to take over; in some respects, therefore, the choice had been limited.

Turning to the question of ventilation, the Authors had referred to the use of a 10-horse-power fan and a 15-inch-diameter duct. Brigadier Westrop thought that 15-inch-diameter ducts were common because steel or aluminium sheets were usually rolled not more than 4 feet wide, but from what he knew of some of the Scottish tunnels he would say that for a tunnel of the size and length in question—the maximum headings, he understood, were at about 2,000 feet—that was about the correct capacity, and the question arose of whether the Authors had used the equipment correctly. They mentioned that they only sucked. If the pipe was brought in up to 200 feet from the face and suction was applied at the same time as quantities of good fresh air were being produced from the tools on the face, that was rather like “eating one’s own tail,” because the fresh air was being drawn out, and the remainder of the tunnel was being cleared only by the inflow from the back. There was sure to be a great deal of experience available in that connexion, and it would be interesting to hear opinions on the desirability of using the equipment in the way in which it had originally been intended that it should be used, namely, to blow

after having first cleared the tunnel by suction. When that was done the air released from the face assisted the air blown from outside in pushing the bad air out of the tunnel. The Authors had used a booster fan to get sufficient air when the line exceeded a certain length. That booster fan, apparently, had not been reversible, and that might be the reason why, having set it one way, they did not change; but what experiments had they carried out on blowing as well as sucking to improve their ventilation?

Brigadier Westrop's third point had already been referred to by Mr Lambert and Mr Brown, namely, the use of the placer. Brigadier Westrop had used a placer on a smaller tunnel farther south in India, and the walls had been so solid that no grouting had been found necessary. He did not believe that the pump would give such a solid effect. Once the placer pipe was buried the concrete was thrown through the pipe with enormous force and everything was filled up, so that no places could be found to grout if the equipment was handled properly.

Brigadier Westrop fully agreed with Mr Lambert that 15 feet could not be accepted as the limiting diameter of the tunnel in which a single shutter could be filled. He had just had a report of some work carried out in America with two placers, where apparently two nozzles, 2 feet 6 inches apart, had been used in the roof of a 40-foot-diameter tunnel, placing about 150 cubic yards an hour. It seemed that the essential point was the speed with which the work was done. With the placer it was necessary to work at great speed, and the arrangements made to supply the concrete to it had to be adequate. If they were, he thought that all the troubles would disappear. The method was fairly new to Great Britain and so was apt to be treated with reserve, but in his view it should become standard practice for almost all tunnel linings. He had used the method and had seen what it could do.

It was possible in India to train men to carry out any operation as efficiently as it could be carried out by British workmen. It was only a question of time. The present Paper, and the film which the Authors had shown when introducing it, indicated what could be done by training. The Indian workman tended to have rather a one-track mind, and it was undesirable to take a man off one job and suddenly put him on another. He was very painstaking and loyal, and would climb into all sorts of dangerous places, seemingly without regard for personal risks.

* * Mr P. O. Wolf asked the Authors to provide additional information which he thought would be required if full advantage was to be taken in future of the many interesting facts and opinions recorded in the Paper.

So far as he knew, this was the first mention in the Proceedings of the Machkund hydro-electric project. Although the main object of the Paper

* * This and the following contribution were submitted in writing upon the closure of the oral discussion.—SEC. I.C.E.

was to deal with a limited number of constructional problems, it would be of interest to have a more complete description of the whole scheme, including possibly some cross-sections of the diversion dam, of the flume channel and aqueduct, of the power station, and of the surge shaft and its gates. The dimensions of some of the works and the final electrical capacity were of the same order as those of some Scottish schemes, for example, Loch Sloy, and engineers would be interested in data enabling them to study the similarities and the differences of treatment.

The value of a record such as the present one lay, to a great extent, in the use which could be made of the information in it by engineers faced with similar problems in future. For that reason the critical observations in the Paper were particularly stimulating. Could the Authors add to them more complete details of the dates of the work and of the conditions under which their contract had to be operated, so that some unusual features might be more fully understood?

Mr C. M. Roberts observed that it was essential that the concrete lining of a tunnel connected directly to a hydro-electric power station should be absolutely sound throughout its thickness. It was not sufficient merely to show a smooth face backed by inferior concrete. Most highland waters had properties which attacked concrete and, in the course of time, any weakness in the concrete resulted in disintegration at the point of weakness. It was inevitable that cracks should form in any continuous concrete lining and such cracks admitted water to the heart of the lining. Any method of placing concrete in the lining which was likely to give rise to weakness in the concrete was not therefore acceptable.

There were three points mentioned in the Paper which seemed to Mr Roberts inevitably to give rise to possible weakness, however efficiently the method was applied:—

- (1) The trailing end of the shutter had to be filled from the open-ended delivery pipe, giving distinct possibilities of segregation.
- (2) At the trailing end, concrete had to slide over the shutter and then drop freely over a considerable height giving further possibilities of segregation.
- (3) When delay occurred during placing with a long sloping surface, it was difficult to reinstate the proper action of the placer without having a run of concrete over the surface of that previously placed and that would inevitably lead to weakness.

For the proper action of the pneumatic placer to be developed it was essential that the end of the delivery pipe should be buried. The sooner that condition could be achieved the better and that was one reason why the pneumatic placer was more suited to small tunnels.

The Authors had stated that honey-combing at the trailing end of the shutter could apparently be eliminated by placing six batches of mortar before placing concrete. Mr Roberts wondered if there was any evidence

to show that, although the concrete surface was good, honey-combing did not exist behind. The extent of honey-combing appeared to be related to the distance which the concrete had to fall, for in tunnels of 9 feet diameter and less the fault was not generally apparent.

The difficulties mentioned above arising from the "intermittent" system of placing could largely be met by placing concrete in the sidewalls in horizontal layers. That could be done by hand placing, the concrete being conveyed to the upper levels of the sidewalls in skips elevated by hoists or ramps and being fed into the shutter direct by hand or through chutes. Alternatively, two small pneumatic placers could be used with shooting "heads" at the ends of the delivery pipes to break the velocity of the concrete. Both placers would travel to and fro to fill the shutter in horizontal layers. Such a method allowed the proper preparation of the upper surface of concrete to receive fresh concrete on top of it. All laitance could be removed and, if a hold-up occurred during placing, there was no question of fresh concrete flowing over the top of the concrete below. When the sidewalls were complete the arch could be placed, using the pneumatic placer with a reduced drop for the concrete, and the end of the delivery pipe could be buried quickly.

The proper action of the pneumatic placer demanded the highest possible rate of delivery of concrete consistent with the capacity of the placer. The highest rates of delivery could usually be obtained by mixing concrete outside the tunnel and conveying it by means of transit mixers.

Figs 4 showed that a small splay was formed in the sidewall above the kerb. Mr Roberts had found that such a splay tended to break off, apparently because of shutter movement and sometimes because of weak concrete in the splay, and that better results were obtained by casting a haunch to the finished profile extending about 12 inches into invert and sidewall before the sidewall was placed.

Could the Authors give details of the size and position of the inspection panels provided in the shutter? They had an influence upon the amount which could be done on the concrete after placing behind the shutters.

Although it would seem that the Authors concluded that vertical sidewalls would improve the finished surface when pneumatic placers were used, such a design could not always be accepted, since it would decrease the hydraulic efficiency of the tunnel and impair the structural value of the lining.

The minimum clearance between rock and the finished concrete surface scaled, from *Fig. 1 (b)*, about 6 inches. That appeared to be the minimum thickness of lining which should be used when mechanical placing was adopted, since a smaller clearance would seriously interfere with the flow of concrete. Mr Roberts asked if the Authors considered that a larger clearance would have been of assistance in obtaining a good quality of concrete in the lining.

The Authors, in reply to Mr Lambert, said that the thickness of the

concrete lining had been specified as 6 inches minimum. In the first tunnel, where the rock had been fairly good, they believed that the final average thickness had been of the order of 20 inches. For the flume tunnel no figure could be given which was in any sense accurate, because in a number of places they had run into difficult ground, as mentioned in the Paper, and there had been one hole in the roof 40 feet high, which naturally had an effect on the average figure!

Mr Lambert had raised the question of the choice of the length of shutter. At that time the Authors believed that the 40-foot length represented all the steel which they were allowed, and that had settled the matter, but they believed that 40 feet was about right. It would have been nice to have had it 80 feet long, but the capital cost of the equipment would have been excessive, since the first shutter had to have a good many additional bits and pieces to convert it for use in the second tunnel. The length of 40 feet made it possible to pour in a reasonable time, but it had had the disadvantage of repeated joints, and they believed that the best answer would be a telescopic form of shutter with continuous placing with a pump or placer. That, however, was a very expensive proposition and would need a very long length of tunnel to justify it.

The concrete had been volumetrically batched on a 1 : 2 : 4 mix.

Several speakers had commented on the pump *versus* placer problem and Mr Brown had remarked that one was on delicate ground in dealing with it. Both machines had many good points and inevitably some bad ones, and the Authors had had to be very careful about what they had said in the Paper to avoid undue bias either way. The operating conditions had to be taken into account. One of the best points about the placer, of course, was the complete (so far as they were concerned) absence of mechanical trouble. The rate of placing was limited only by the rate at which one could put concrete into it and by the compressor-capacity available to pump the concrete out. But the cost of operation was quite considerable if the cost of the compressed air was high, and that was also a factor to be taken into account. They thought that in the conditions in which they had been working a concrete-pump might have suffered serious mechanical trouble and it would have been impossible for them to manufacture spare parts at site for a machine of that kind, and it was doubtful if spares could have been stocked to meet every contingency.

They had been glad to learn from Mr Tozer that a new form of platform had been introduced. There was no denying the advantage of simultaneous drilling and mucking in reducing the cycle time, and the whole of the equipment was much lighter and less costly than that required in the older techniques, so that the new technique seemed to be the answer. Particularly in the second tunnel, if it had been possible to take a piece of pipe which was a little longer than the standard length to bridge a gap, that seemed to be a very effective way of dealing with the difficulty where excessive overbreak had occurred.

Mr Brown had suggested that the size of the tunnel was not clear. It was true that the Authors had not given the thickness of concrete, but the internal size of the linings were given in *Figs 1 (b)*. The concrete thickness had, as mentioned above, been designed as a minimum of 6 inches. Mr Brown's conclusion that the average progress figure had been about 80 feet per week was correct, but that was an average figure obtained after the men had been reasonably trained; it had taken a long time to get them up to that figure. Their best week, when everything went right, was 130 feet, but figures of 80 or 90 feet in a week were regularly obtained in the second tunnel, the week being the same as that normally worked in Britain. Only maintenance work was done on Sundays; sometimes the Saturday shift worked on a little to finish a round, but normally on Saturday night and Sunday no actual tunnelling work had been in progress.

Mr Brown had expressed interest in the fact that the 1-yard skips had been found to be too light, and Brigadier Westrop had explained that for certain items of plant there had been no choice. The skips had been one of them. They were Indian-made wagons, similar to those used in the United Kingdom, but using a somewhat lighter gauge of metal and lacking some of the stiffening. The Authors agreed that a 2-yard skip would have been much better if sufficiently strong to stand the rough treatment and rough loading from a rocker-shovel.

They had had little or no trouble with the Diesel locomotives. They had referred to the care taken in the maintenance of those machines, and one man alone had been responsible for filling them with fuel, to make sure that it was clean. The atomizers and fuel pumps had been kept in first-class condition. In those circumstances the smoke was nil and the smell negligible. In those relatively large tunnels it had not been anything like so serious as the smell of explosives, which had worried them at times.

Mr Brown had also raised the question of the use of internal vibrators. There had been time between the actual shootings of the concrete for men to use internal vibrators; they had kept the inspection panels open and a certain amount of hand punning had been done in the intervals between shots; but the ordinary pencil type of vibrator had tended to bore its way down the shutter and form a very local point of vibration. That had tended to draw the "fat" from the concrete to make a circle round the vibrator, and the effect had not spread throughout the shutter. Better results had been obtained with heavy insertion-type vibrators used externally by mounting them in suitable clamps fitted to the main ribs of the shutter, moving them from one position to another following the freshly poured concrete.

Brigadier Westrop had raised the question of experiments in ventilation. In the earlier stages, when they had been working with only the external fans which had reversing ducts, they had carried out experi-

ments, but the time had come when with the use of the external fan only on a long length of pipe, something of the order of 750 feet in, its effectiveness had been very much reduced and it had been necessary to install booster fans. The only fans available were one-way axial propeller-type and it was obviously impracticable to change them round after each blast to either blowing or sucking. The final conclusion reached on the basis of experiment had been that they were better placed on suction. It had been found that when blowing fresh air into the face (it did not matter much whether it came from the compressed air main or the blower duct) it tended to churn up with the existing fumes, and they had obtained a weaker and weaker mixture, not a pocket of clean air. By blowing compressed air at the face the same effect had occurred but gradually, with the suction duct working 200 feet away, the face had tended to clear and the "mixture" had been gradually drawn away. They agreed that they had been "eating their own tail" to a certain extent, but the method had ultimately cleared the atmosphere 200 feet from the face, which enabled the men to work. The fumes remained down the tunnel, and the loco drivers had to pass through them, but the period involved was short and it seemed to be the best way of dealing with the problem in that particular case.

It was very interesting to learn that placers were being used in America on a tunnel 40 feet in diameter. Perhaps Mr Lambert had put his finger on the real point, which was that the materials which the Authors had had to use, with the lack of sufficient good river sand and the limited facilities for getting the various grades screened out from the crushers and then correctly re-proportioned, had been the cause of the difficulty. In other words, if it were possible to get the concrete correctly graded for the placer it would, apparently, be possible to use a placer on any size of tunnel, but, considering the materials and equipment which the Authors had had to use, the Paper had given their conclusions quite correctly so far as the Machkund Tunnels were concerned.

The Authors agreed with Brigadier Westrop's remarks about the training of Indians. They could not have wished for men who were more willing and more responsive to training or who took a greater pride in their job. The only fundamental difference between India and Great Britain, once one had got the right group together, was that one needed more men, but that was entirely a question of physique and not a lack of willingness.

In reply to Mr Lambert's request for further data about the rate of placing, that had been limited in the Machkund tunnels, not by the capacity of the placer, but by the output of concrete from the mixer. A 1-cubic-yard mixer had been used and from that they had averaged twenty mixes per hour but not a full cubic yard per mix. They had found that they had to cut down the volume of each mix slightly, both to suit the size of the trucks they had had available for bringing in the aggregate

and also because they had found that it did not pay to try and fill the placer too full. The result was that the mixer had turned out an average of 15 cubic yards per hour, which was well within the capacity of the placer.

The average amount of concrete per length of shutter was about 100-110 cubic yards, as Mr Lambert had calculated, so the pouring time was 7 to 8 hours. They had found it was possible to start concreting 4 hours after the start of moving the shutter, which gave roughly 12 hours from the end of setting time to completing a length of concrete. The cycle was, therefore, 28 hours, including a 16-hour setting time, and so five lengths a week thus fitted very nicely into the 6-day working week.

Mr Tozer, the Authors thought, had made a very interesting point about the use of the platform having cut down the effective drilling time from that required for one man to drill eight holes to that required to drill four holes only. It would be interesting to take it a stage further and see if it was possible, by speeding up the mucking, which had been slow in relation to the size of the tunnels, to eliminate the mucking time from the effective time-cycle. The cycle time would then consist essentially only of the total times for fume clearance, drilling (being the time taken for one man to drill eight holes), and loading and blasting; the mucking and drilling of the bottom holes would be done whilst the cut and easers were being drilled. In view of the possibilities of that overlapping technique, it was, therefore, very interesting to hear that experiments were being made with the object of producing a simpler and more adaptable platform, which would answer the criticisms put forward.

In reply to Mr Wolf, the Authors regretted that it had not been possible to include a full description of the whole project; they were hardly qualified to do so since their activities had been confined to the construction of the two tunnels. If Mr Wolf was very anxious to have details of the other works, the Authors were sure that they would gladly be supplied on request by the Chief Engineer to the Electricity Department, Madras.

The conclusions reached by Mr Roberts in regard to the use of the pneumatic placer were clearly similar to those of the Authors. At several places, subsequent examination had disclosed concealed honeycomb in the lining, but that had been an exception rather than the rule. The discussion had emphasized the importance of correctly graded aggregates and rapid placing, conditions which had not been fully realized at Machkund, and the Authors were prepared to accept the evidence put forward that, given the right conditions, satisfactory linings could be placed in large tunnels with a pneumatic placer.

The Authors fully agreed with Mr Roberts's comments regarding the splay at the foot of the sidewall shutters and had referred to the matter on p. 135.

Inspection panels were provided in the shutter throughout its full length at two levels on both sides. The first line was about half-way up the sidewall and the second about half-way between the springing and crown of the arch.

The Authors had not intended to convey the impression that they would prefer to design tunnels with vertical sidewalls in the interests of an improved finish. The trouble experienced with "water marks" was described in the Paper and the Authors merely justified their theory as to the cause by the fact that they had rarely occurred in the second tunnel, where the sidewalls were vertical. The defect may have been peculiar to the conditions obtaining at Machkund.

The Authors agreed with Mr Roberts that small clearances seriously interfered with the flow of concrete when mechanical placing was being used but that could be offset by increased punning in the areas affected. In those tunnels, closely supervised hand punning had been used, and it had been found that the tendency to produce segregation or honey-combing had not been noticeably greater where the lining was relatively thin.

CONFERENCE ON THE NORTH SEA FLOODS OF 31 JANUARY/1 FEBRUARY, 1953

On Wednesday and Thursday, 16 and 17 December, 1953, a Conference was held at the Institution to discuss the various aspects of the disastrous floods arising from the storm surge in the North Sea during the night of 31 January/1 February, 1953. The discussions ranged from the meteorological conditions causing the surge to the building of permanent coast defences designed to withstand the effects of recurrence of the same or greater magnitude.

The following twelve Papers were presented for discussion :—

- “ A General Survey of the Damage Done and Action Taken,” by J. V. Spalding, B.A., M.I.C.E.
- “ Storm Surges on the East Coast of England,” by Commander W. I. Farquharson, F.R.I.C.S., R.N.(Ret.).
- “ Some Meteorological Aspects of North Sea Floods, with Special Reference to February 1953,” by S. P. Peters, B.Sc.
- “ Soil Mechanics Studies of Failures in the Sea Defence Banks of Essex and Kent,” by L. F. Cooling, D.Sc., and Arthur Marsland, M.Sc., Grad.I.C.E.
- “ Design of Sea Defence Works in Relation to Height of Tide and Degree of Exposure,” by C. H. Dobbie, B.Sc.(Eng.), M.I.C.E.
- “ The Incidence of Storm Surges as a Factor in the Design of Coast Protection Works,” by Jack Duvivier, B.Sc.(Eng.), M.I.C.E.
- “ Sea Defence Works on a Sandy Eroding Coast with Scanty Littoral Drift,” by S. W. Mobbs.
- “ Damage to the Kent Coastline, and Restoration Works,” by G. Cubley Crowther, O.B.E.
- “ Damage to the Essex Coastline, and Restoration Works,” by E. L. Snell, O.B.E., A.M.I.C.E.
- “ Sea Defences in the Wash and Estuary of the Great Ouse in Relation to the Tidal Surge of the 31st January, 1953,” by W. E. Doran, O.B.E., B.A., B.A.I., M.I.C.E.
- “ Flood Damage in Norfolk and Suffolk,” by K. E. Cotton.
- “ Damage and Remedial Operations on the Lincolnshire Coast,” by F. H. Tomes, O.B.E., M.I.C.E.

Amongst those present were a number of eminent engineers from Holland and Belgium, some of whom contributed to the discussions. The Dutch delegation included :—

Dr Ir J. A. Ringers, formerly Minister of Public Works and Reconstruction, President of Koninklijk Instituut van Ingenieurs ;

Professor Ir J. Th. Thijsse, Professor at the Technological University of Delft and Director of the Laboratory for Hydraulic Research at Delft ;

Professor Dr P. Groen, Assistant Director of the Oceanography and Marine Meteorology Section of the Royal Meteorological Institute of the Netherlands at De Bilt and Extra-ordinary Professor at the Free University of Amsterdam ;

Dr Ir J. van Veen, Chief Engineer of Hydraulic Affairs (Rijkswaterstaat) and Secretary to the Delta Commission ;

Dr K. R. Postma, Director of the Weather Service and Aeronautical Meteorology Section of the Royal Meteorological Institute ;

Ir R. Ver Loren van Themaat, member of the Delta Commission ;

Ir H. Sangster, Secretary of Koninklijk Instituut van Ingenieurs ;

Ir P. J. Wemelsfelder, Chief Engineer of Hydraulic Affairs.

The Belgium delegation included :—

Mr J. Blockmans, Inspector-General, Maritime Services, Antwerp ;

Mr J. Verschave, Chief Engineer, Director of Civil Engineering Department, Belgian Coast.

A report of the Conference, including the twelve Papers and abstracts of discussions will be published by the Institution in a separate volume, price 15s. 0d. to members and £1 2s. 6d. to non-members, post free. It is hoped that this will be available in May.

ELECTION OF MEMBERS AND ASSOCIATE MEMBERS

The Council, at their meetings on the 20th October 1953, 17th November, 1953, 15th December, 1953, and 16th January, 1954, in accordance with By-law 14, declared that the undermentioned had been duly elected as :—

MEMBERS

Home

LIND, HERMAN PETER THYGESSEN, B.Sc. (Copenhagen).
MOLLER, AXEL, R.D., B.Sc. (Copenhagen).

Abroad

SEMENZA, CARLO.
VILLARES, ARNOLD DUMONT, O.B.E.

ASSOCIATE MEMBERS

Home

AIRD, IAN ADAM, B.C.E. (Melbourne).
ARGENT, HERBERT WILLIAM, Grad. I.C.E.
AXTELL, ROGER JOHN, B.Sc.(Eng.) (London), Grad.I.C.E.
BAILEY, JOSEPH IAN, B.Sc. (Leeds), Stud.I.C.E.
BARR, DAVID IAN HUNTER, Stud.I.C.E.
BEDFORD, THOMAS EDWIN, B.Sc. Tech. (Manchester).
BEECHING, DENIS, B.Sc.(Eng.) (London).
BENNETT, VIVIAN WILFRID, Grad.I.C.E.
BERTIE, JOHN MICHAEL.
BRAINT, ALAN KEITH, B.Sc.(Eng.) (London), Grad.I.C.E.
BRAMWELL, DAVID.
BRATTLE, JOHN DOUGLAS, B.Sc.(Eng.) (London).
BROOKBANK, KENNETH, B.Sc.Tech. (Manchester).
BROWN, JOHN THOMPSON, B.Sc. (Durham).
BROWN, PHILIP ANTHONY PIGGOTT, B.A., B.A.Sc. (British Columbia).
BUCHANAN, ANGUS, B.Sc. (Edinburgh).
BUCHANAN, JOHN McROBERTS, B.Sc. (Belfast).
BUGDEN, GEORGE PHILIP, M.A. (Cantab.).
CAMPBELL, JAMES GRAY, B.Sc. (Belfast).
CARMICHAEL, WILLIAM FLEMING.
CARRICK, EDWIN SOOTE, B.Sc. (St Andrews).
CHARNOCK, DENYS BURRELL, B.Eng. (Liverpool), Grad.I.C.E.
CHEESBROUGH, ROBERT, B.Sc.Tech. (Manchester), Grad.I.C.E.
CHIN, HUBERT.
CLAY, GEOFFREY ARTHUR VERNON.
CLAYTON, THOMAS ROY, B.Sc.(Eng.) (London).
CLUTTERBUCK, PETER ROYSTON, Grad. I.C.E.
COLBRIDGE, GEORGE BRIAN, B.Sc.(Eng.) (London).
COLLINS, SYDNEY PATRICK, B.Sc.Tech. (Manchester), Stud.I.C.E.
COOPER, NEVILLE THOMAS, B.Sc.(Eng.) (London).
CROSSLEY, THOMAS RAYMOND.
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Paper No. 5950

“The Variability of Concrete and its Effect on Mix Design”

by

Francis Robert Himsworth, B.Sc., Ph.D.

(Ordered by the Council to be published with written discussion.)†

SYNOPSIS

The current procedure used in the design of concrete mixes to give a certain minimum strength involves the assumption that the variability increases as the mean strength increases, the coefficient of variation remaining constant. This assumption is examined and is concluded to be false, the variability being independent of the mean strength, other things being equal. This necessitates a radical change in the design procedure; the data necessary for the proposed procedure are given.

The causes of variation in concrete strength are analysed in detail. It is shown that nothing is to be gained from extreme accuracy in weighing the ingredients. Several official and unofficial publications purport to show that cement-quality variations account for some 50 per cent of the total variation in concrete strength. This is shown to be false, being based on faulty analysis of the data; correct analysis shows the cement contribution to be only about 10 per cent even when other sources of variation are well controlled. It appears that control of workability at the mixer, in conjunction with reasonable care in proportioning, gives more effective control over strength than is possible by accurate proportioning and by estimation of the moisture content of the aggregates. The effects of factors other than those controlled by the designer and the mixer operator are summarized.

INTRODUCTION

IN recent years the design of concrete mixes to achieve the required workability and strength with maximum economy has been put on a scientific basis. It is now a recognized, though not yet common, practice to employ a properly designed mix, together with careful control of the batching and mixing operation, in place of the formerly accepted method of stipulating, say a 1 : 2 : 4 mix, irrespective of the aggregates used and of the properties required. An important feature of modern design is the recognition that concrete, however made, is variable in its properties, particularly strength, and that in the design procedure account must be taken of the variability as well as of the average strength. If it is required that none, or very little, of the concrete should have a strength below some specified value, then, because of the variability, the average must be made appreciably higher than this minimum; the greater the variability the greater the gap between

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average and minimum. It may be noted at this stage that it is not possible to fix any definite minimum such that no test results will be lower; any such minimum would have to be set so low as to be valueless in design. By "minimum" strength, therefore, is meant a strength below which very few, if any, results will fall; for consistency with other published works, the number of results expected to fall below the minimum is taken as 1 per cent.

This Paper is presented in two Parts. In Part 1 the statistical principles used in concrete mix design are outlined, and a radical change in the procedure for calculating the average strength required to ensure the specified minimum is proposed. Part 2 contains a detailed analysis of the factors causing variation in concrete strength, particularly the water/cement ratio. From this analysis are drawn several conclusions of practical importance to concrete engineers, for example, the effects of batching errors, with a comparison of weight and volume batching, and of control of water/cement ratio by assessment of workability. It is shown that cement-quality variation has in the past been estimated by faulty methods, leading to gross overestimates of its effect on concrete strength.

PART 1

STATISTICAL PRINCIPLES USED IN DESIGN

Modern concrete design involves certain statistical principles, which are summarized in this Section.

Normal Distribution

When concrete is being made from uniform stocks of cement, aggregates, and water, and control of proportioning, mixing, and curing, whether good or bad, is uniform, test cubes, each made from one batch of concrete, rarely exhibit identical strengths. The variation may amount to plus or minus several hundred lb. per square inch about the mean value. The extent of this variation depends on, amongst other things, the care taken in proportioning. If a large number of such cubes are made under reasonable uniform conditions and the test results are divided into groups having a range of, say 50 lb. per square inch, then a plot of the numbers of results falling in the various groups (see *Fig. 1*) will be found to lie roughly on a bell-shaped curve. Most of the results lie relatively close to the mean strength, but there are a few scattered wide on each side.

Results of concrete tests, made under carefully controlled uniform conditions give a curve closely resembling a theoretical curve widely used in statistical calculations. This is the co-called "normal" curve. It is defined by the equation:

$$y = \left(\frac{1}{\sigma\sqrt{2\pi}} \right) e^{-\frac{(x-\bar{x})^2}{2\sigma^2}} \cdot dx \quad (1)$$

x denotes observed result

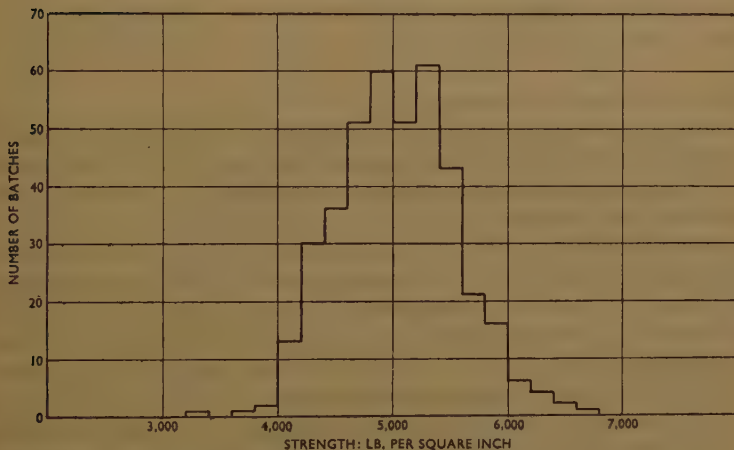
 $\bar{x}$.. mean of all results

y ,, relative frequency with which the observations lie in the range x to $x + dx$

σ ,, standard deviation, which is defined by the equation :

[illegible]

Fig. 1



HISTOGRAM FOR CONCRETE STRENGTH AT 28 DAYS

where x and $\bar{x}$ are as defined above, N denotes the total number of observations, and Σ denotes summation over all values of x . V is known as the "variance," and is, except that the divisor is $(N - 1)$ instead of N , the mean square deviation from the mean value. The standard deviation is the square root of the variance, that is, the root-mean-square deviation.

The standard deviation is a measure of the degree of spread of the results; a high value indicates very variable results, whilst a low value indicates relatively constant results. Other quantities may be used to express variability, for example, the range (maximum–minimum) of all the results, but the standard deviation is the most reliable of these measures, and should be used wherever possible.

It will be seen from equation (1) that if the data can be assumed to be "normally" distributed, and if the mean, $\bar{x}$, and the standard deviation, σ , be known, then the probability that x will fall in any given range can be calculated. Alternatively, a probability, say 1 per cent, can be assumed and the corresponding value of x , say x_0 , can be calculated; out of a large number of results, only 1 per cent will fall below the value x_0 . For

"normally" distributed data, 1 per cent of the results will, in general, be less than x_0 , where :

$$x_0 = x - 2.33\sigma \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Factors corresponding to 2.33 for various probabilities are :

Probability	Factor
2.5 per cent	1.96
1.0 "	2.33
0.1 "	3.09 etc.

The symbol for the standard deviation is usually written σ for an accurately known value, or s for a measured value. For simplicity, s will be used throughout the Paper.

Application to Concrete Design

The application to concrete design is as follows. The strength required having been calculated, it must be ensured that little, if any, of the concrete placed has a strength lower than this calculated value. It is unreasonable to stipulate that none shall have a lower strength, but the small figure of 1 per cent commonly specified gives ample safety margin. A few designers depart from that figure but it is the most usual and will be used throughout this Paper. It should be emphasized, however, that it is only when considering large numbers of tests that 1 per cent of results will fall below this minimum value ; in a set of 100 the percentage might very well be zero, or 5. Having stipulated that not more than 1 per cent of the results should fall below x_0 , the "minimum" strength, the mean strength for which the concrete must be designed can be calculated from equation (3). For example, if the minimum is 3,000 lb. per square inch, and the standard deviation is expected to be about 800 lb. per square inch, then :

$$\bar{x} = 3,000 + 2.33 \times 800 = 3,000 + 1,864 = 4,864 \text{ lb./sq. in.}$$

Coefficient of Variation

Another measure of variability, closely allied to the standard deviation, is the Coefficient of Variation, defined as :

$$\text{C.V.} = v = 100 s/\bar{x} \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

It is thus the standard deviation expressed as a percentage of the mean. In some circumstances it is a useful and appropriate measure of variability. For example, if the error of a testing machine is greater when the strength of the test specimen is high, in particular if the error is proportional to the load, then the coefficient of variation—in this case a measure of the testing error—will be the same whatever the actual strength being measured. The standard deviation, on the other hand, will not be constant. The coefficient of variation would thus be a single measure of variability, characteristic of the machine, for any part of the range. If, however, the magnitude of the error is the same for all parts of the range the standard deviation is the appropriate measure of testing error.

Standard Deviation and Coefficient of Variation in Concrete Testing

It is common practice ^{1, 3, 6, 8} to use the coefficient of variation as a measure of the variability of concrete made according to a given system of control. It is thus assumed that a given value will be achieved whatever the mean strength of the concrete made; if the coefficient of variation is 15 per cent, then, if the mean strength is 2,000 lb. per square inch, the standard deviation will be 300 lb. per square inch; but if, other things remaining equal, the mix is altered to give a mean strength of 6,000 lb. per square inch, the coefficient of variation will remain at 15 per cent, and the standard deviation will thus be 900 lb. per square inch.

It seems to have been assumed that this is the case; the variability of concrete is commonly expressed as a coefficient of variation, without reference to the mean, and the current design procedure is based on this assumption. So long as only a narrow range of concrete strength is considered (for example road concrete, usually designed for 5,000 lb. per square inch), it is immaterial which is used as the typical measure of variation.

In general civil engineering work, however, different situations call for widely different strengths, and it is important to know whether or not the coefficient of variation remains constant, other things being equal, over a wide range of mean strength. Available data do not justify the assumption that it is independent of mean strength. It is, *a priori*, just as plausible an alternative assumption that the standard deviation is unaffected by a change of mean strength. A main object of this Paper is to examine the evidence for these two hypotheses, and the effect on concrete design of adopting one or the other.

The Combined Effect of Several Sources of Variation

Suppose the strength of concrete to be affected by two or more separate causes of variation, say cement-quality variation and variation in water/cement ratio. If only one cause, cement variation, operates, the strength variation can be expressed by the standard deviation, s_1 . If only the other cause operates, the resulting standard deviation is s_2 . It is required to find the standard deviation of the concrete strength when both causes operate together. It is not, as is frequently assumed, $(s_1 + s_2)$; the correct expression is:

$$s^2 = s_1^2 + s_2^2$$

or

$$V = V_1 + V_2$$

that is

$$s = \sqrt{s_1^2 + s_2^2}$$

The same principle governs the addition of ranges or "total variations." The range is approximately a given multiple of the standard deviation, and if two or more ranges, associated with different independent sources of variation are to be combined, the correct estimate is, as for the standard deviation, the square root of the sum of the squares. The literature on

¹ The references are given on p. 194.

concrete contains many examples of the wrong method ; the individual "total variations" are added to give the overall "total variation." This leads to serious miscalculations.

"Total Variation" as a Measure of Variability

It is common practice to express the variability of concrete, estimated from a number of batch strengths, as the "total variation" or "percentage total variation." The "total variation" is better described as the "range."

The range is often, and properly, used as a measure of variability, when a number of small groups are involved ; the mean of several ranges can give a reliable measure of variability. The number of groups should be twenty or more, and the number of observations in each group, preferably not more than about six, and certainly not more than twelve. Otherwise the range is an unreliable measure ; that is to say, it may vary considerably in two sets of observations, even though the true variability is the same for both. This is because the range is entirely governed by the extreme observations, and in sampling, say, a "normal" population it is always possible that an unusually divergent result may be derived, giving an apparently large variability. The use of the range based on one set of results is, therefore, inadvisable. In order to measure the degree of variation, the standard deviation, which is the most efficient measure of variation should be calculated. To quote only the range of a considerable number of results is equivalent to discounting much of the information on variability contained in the results.

If the standard deviation is known, then it is quite justifiable to calculate from it the probable range of a set of results ; for example, to state that out of 100 results, "normally" distributed, only one or two are likely to lie outside the range $\bar{x} \pm 2.33s$. But this is only an expected result ; the actual number may well be zero, or say 5. To carry out the reverse calculation, that is, to state that the range of 100 results is 2,000 lb. per square inch, and hence estimate the standard deviation is inadvisable, and gives an uncertain estimate. The correctly calculated standard deviation is, on the other hand, a reliable estimate of the true value if 100 results are available.

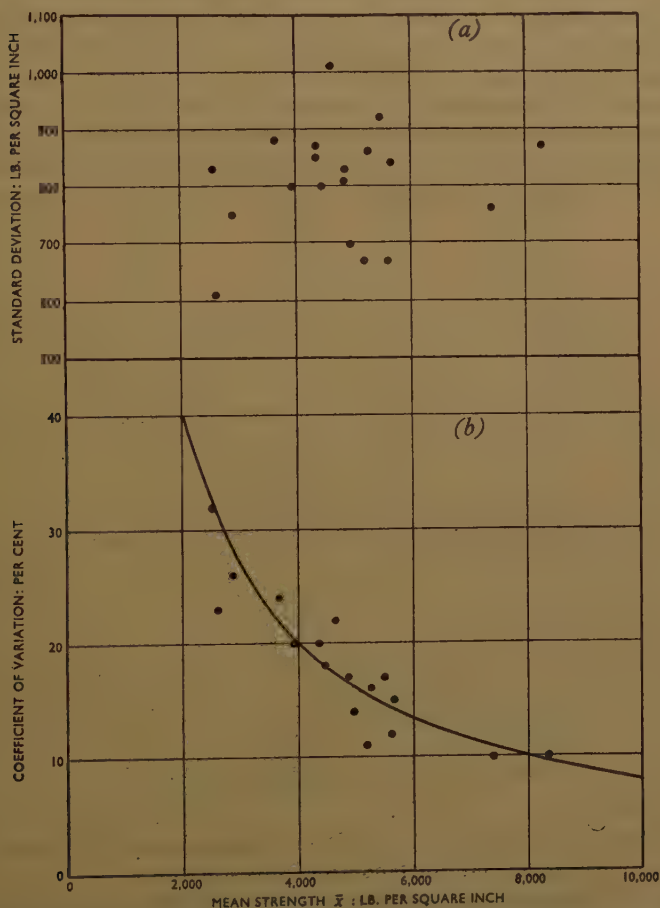
It is strongly suggested that the practice of quoting the range (under any name) should be abandoned, and the standard deviation should be calculated in all cases. Strength data are not acquired without considerable expense, and it is foolish to discard part of the information they contain.

EVIDENCE ON THE RELATIONSHIP BETWEEN MEAN AND STANDARD DEVIATION

It is not easy to find large bodies of reliable data from which to estimate the relationship between mean strength and standard deviation. The ideal

would be to have considerable numbers of test results from a site at which concretes of widely different mean strength were being made under identical systems of control and supervision. If such data could be accumulated for various systems of control they would be of the greatest assistance in

Fig. 2



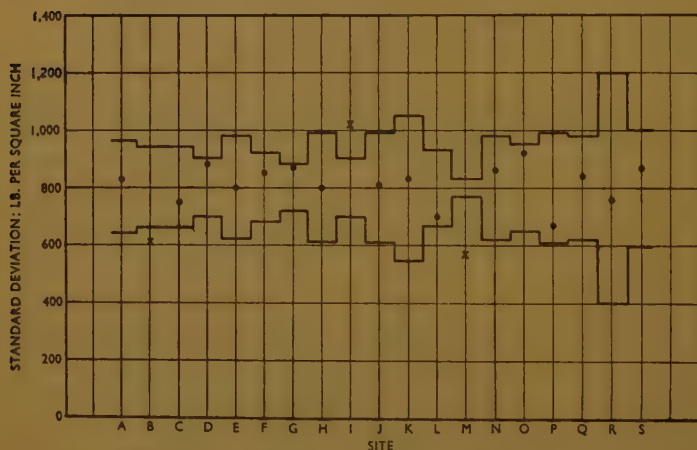
STANDARD DEVIATION AND COEFFICIENT OF VARIATION PLOTTED AGAINST
MEAN STRENGTH

setting up a rational method of concrete design. Meantime, however, there exist in the literature sufficient data to justify a review of current design procedure. Attention will be confined here to examining whether the data support the hypothesis that for a given level of control, the coefficient of variation is independent of the mean, or the hypothesis that the

standard deviation is independent of the mean. It is possible, of course, that neither is true, but this is a refinement, as will appear.

Two useful collections of results are quoted by Sparkes⁵ and by the Road Research Laboratory.² Sparkes shows, in his Fig. 1, the coefficient of variation plotted against the mean, for nineteen jobs. The Laboratory give a similar graph,² but instead of the standard deviation, the "percentage total variation" is plotted, that is, half the difference between the maximum and minimum result, expressed as a percentage of the mean. This measure was used because the coefficient of variation could not be calculated in all cases from the information available. It is roughly a constant multiple of the coefficient of variation if the numbers of results for each point are about the same. It could reasonably be $2-2\frac{1}{2}$ times the coefficient of variation. Although the degree of control was not the same on

Fig. 3



CONTROL CHART FOR STANDARD DEVIATION

all the sites, these data are of particular value in that they do represent practical conditions, and not laboratory investigations.

Mr F. N. Sparkes has very kindly made available the data on which both curves were based. These are given in Table 1. There are twenty-five sets of results; for nineteen of them the standard deviation was calculated, but in six cases only the range had been recorded. For the latter, a rough estimate of the standard deviation has been made, based on the range and the number of results; these are shown bracketed, and it should be realized that they are subject to a considerable uncertainty. There is no apparent trend in the standard deviation, but the coefficient of variation falls steadily as the mean strength increases. Fig. 2 shows (a)

TABLE 1.—MEAN AND STANDARD DEVIATION OF CONCRETE STRENGTH

Site	No. of tests	Mean strength : lb./sq. in.	Standard deviation : lb./sq. in.	Range : lb./sq. in.	Coefficient of variation : per cent
A	64	2,550	830	3,800	32
B	91	2,600	610	2,890	23
C	85	2,870	750	3,210	26
D	169	3,640	880	4,400	24
E	50	3,930	800	4,280	20
F	114	4,340	850	3,690	20
G	231	4,340	870	4,240	20
H	48	4,460	800	4,150	18
I	167	4,630	1,010	3,680	22
J	48	4,820	810	3,140	17
K	28	4,850	830	3,500	17
L	95	4,960	700	3,500	14
M	1,614	5,170	570	3,950	11
N	48	5,230	860	3,360	16
O	72	5,460	920	3,800	17
P	48	5,600	670	3,400	12
Q	54	5,630	840	3,600	15
R	23	7,400	760	2,800	10
S	44	8,300	870	3,280	10
T	900	3,720	(680)	4,450	(18)
U	718	4,000	(360)	2,150	(9)
V	150	4,100	(710)	3,840	(17)
W	150	4,680	(710)	3,840	(15)
X	986	5,350	(700)	4,640	(13)
Y	56	6,100	(850)	3,890	(14)
Average (A—S)			802		

standard deviation, and (b) the coefficient of variation plotted against mean strength. Allowing for some scatter arising from differences in degree of control, and from the rather small numbers of tests in some of the sets, it may be concluded that the standard deviation is independent of the mean strength. The (unweighted) average is 802 lb. per square inch. The curve at the bottom of *Fig. 2* corresponds to a constant standard deviation of 800 lb. per square inch. The observed points clearly cluster round this line.

Fig. 3 is a statistical control chart for standard deviation, using a mean value of 800 lb. per square inch; the limits are those appropriate to the number of observations represented by each point for a probability of 1 per cent.* It is evident that the data, with the exception of three points, are consistent with a uniform standard deviation of about 800 lb. per square inch, and it seems reasonable to assume that on sixteen of the

* Control limits = $800 \left(1 \pm \frac{2.33}{\sqrt{2n}} \right)$; n = number of observations.

nineteen sites the level of control of batching gave a value of about 800 lb. per square inch ; on one, control was worse, and on two it was better.

In Table II of the A.S.T.M. "Report on Significance of Tests of Concrete and Aggregates,"⁸ the results of one hundred tests each on 1 : 7, 1 : 5, 1 : 4, and 1 : 3 concretes are summarized, the average strength and coefficient of variation being quoted. These figures are given in Table 2 below, together with the standard deviations calculated from them.

TABLE 2.—AVERAGE STRENGTH AND STANDARD DEVIATION OF CONCRETE (A.S.T.M.)

Mix	Number of tests	Average compressive strength : lb./sq. in.	Coefficient of variation : per cent	Standard deviation : lb./sq. in.
1 : 7	100	1,784	12.88	230
1 : 5	100	2,566	10.74	276
1 : 4	100	3,122	8.57	268
1 : 3	100	3,504	7.75	272
Average				*262

* Root mean square.

It is apparent from inspection that the coefficient of variation decreases steadily as the mean increases ; the standard deviation on the other hand remains practically constant, with no trend. It can be shown that there is no real evidence of variation in the standard deviations ; the maximum difference from the average value is 32 lb. per square inch, which is negligible ; it could well arise from chance variations if all four values were estimates of a common value. The data of Table 2 thus strongly support the hypothesis that the standard deviation is the same (other things being equal) whatever the mean strength.

Other data, covering narrower ranges of mean strength, support the conclusion that the standard deviation is not dependent on the mean.

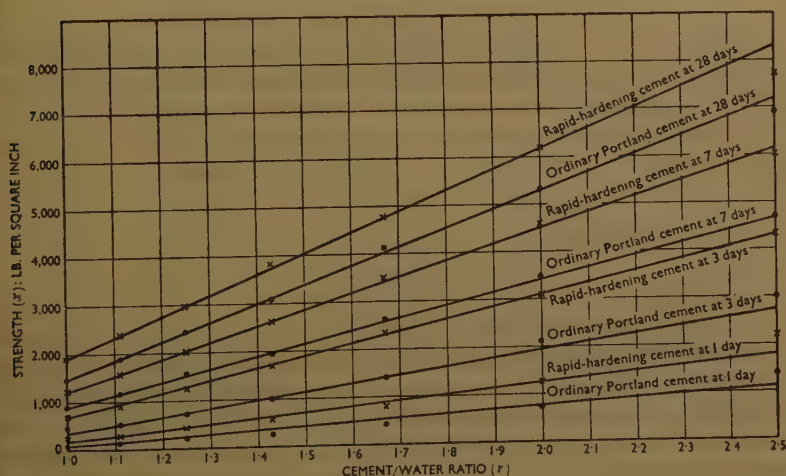
The published evidence, therefore, supports the hypothesis that, other things being equal, the standard deviation of concrete strength is independent of the mean strength, within limits wide enough to cover all cases of practical importance.

An exception to this general statement should be noted ; it relates to the variability of concrete tested at different ages. The strength increases with time in the manner shown by the curves on *Fig. 1* in reference 3 in which strength is plotted against water/cement ratio for ages of 1, 3, 7, and 28 days, and for 3 months, and 1 year. It has been found that the diagram can be considerably simplified if, instead of the water/cement ratio, the cement/water ratio is plotted, since this gives a straight-line graph, up to a cement/water ratio of 2.5, apart from deviations at very high strengths

—more than 8,000 lb. per square inch—in the case of the 3-months and 1-year graphs. The fitted straight lines and observed points—scaled from *Fig. 1* of Road Note 4—for 1, 3, 7, and 28 days are shown in *Fig. 4*. For strengths up to 7,000 lb. per square inch the fit is very good indeed, and it is suggested that the cement/water ratio, denoted by r , should be used in place of the water/cement ratio, because this gives a simpler graph, from which it is much easier to read off the ratio required to achieve a given strength.

The effect of inaccuracy in cement/water ratio on concrete strength is discussed fully in Part 2 of this Paper. If the actual ratio varies around the target figure, with a standard deviation $s(r)$ say, then the resulting

Fig. 4



RELATION BETWEEN CEMENT/WATER RATIO AND STRENGTH

variation in concrete strength, assuming for the moment that only variation in r causes the strength to vary, is given by :

$$s(x) = s(r) \frac{\delta x}{\delta r}$$

It has been seen that $\delta x / \delta r$ is constant at a given age, and so therefore is $s(x)$. But $\delta x / \delta r$, and hence $s(x)$ are different at different ages.

The values of $\delta x / \delta r$ for ordinary Portland cement at ages of 3 days to 1 year are as follows :

Age when tested	$\delta x / \delta r$ (lb./sq. in. per unit change in r)
3 days	1,790
7 "	2,570
28 "	3,690
3 months	4,600
1 year	6,350

The variability thus increases as the age increases, that is, for a given concrete mix, as the strength increases. This does not, however, invalidate the statement made in the above-quoted hypothesis, which refers to different concrete mixes tested at a given age.

The values of $\delta x/\delta r$ for a given value of r are closely proportional to the strength corresponding to that value of r . In other words, for a given mean value of r , the nature of the variation is such that the coefficient of variation is the same at all ages. This is easily seen to be the case :

$$s(x) = s(r) \delta x/\delta r$$

But $\delta x/\delta r$ is proportional to $\bar{x}$, the mean strength ; now writing kx for $\delta x/\delta r$ gives :

$$s(x) = k\bar{x} s(r)$$

$$\text{C.V.} = 100 s(x)/\bar{x} = 100 ks(r) = \text{constant}$$

As is shown in later sections, variation in cement/water ratio is only one factor contributing to concrete-strength variation, and it does not follow that the overall coefficient of variation is the same at all ages.

The effect of the conclusion given in the above-mentioned hypothesis on the design of concrete mixes should now be examined.

CALCULATION OF MEAN STRENGTH TO WHICH CONCRETE MUST BE DESIGNED

Current Method

The method in current use 1, 3, etc., will be described first. A certain "minimum" strength is specified ; this is usually taken to be that strength below which, in the long run, not more than 1 per cent of batch strengths would fall. The strengths are assumed to be "normally" distributed, an assumption which appears to be reasonably close to the truth. The current method assumes that, for a given level of control, the coefficient of variation is independent of the mean strength.

For a given mean strength, $\bar{x}$, and standard deviation, s , the value below which 1 per cent of the strengths will, in the long run, lie, is given the equation for "minimum strength" :

$$x_0 = \bar{x} - 2.33s \quad . \quad . \quad . \quad . \quad . \quad (5)$$

Since

$$\text{C.V.} = v = 100 s/\bar{x}$$

$$\therefore s = v\bar{x}/100$$

$$\begin{aligned} \therefore x_0 &= \bar{x} - 0.0233v \bar{x} \\ &= \bar{x} (1 - 0.0233v) \end{aligned}$$

$$\bar{x} = x_0/(1 - 0.0233v) \quad . \quad . \quad . \quad . \quad . \quad (6)$$

The concrete is designed to give a strength $\bar{x}$, as calculated from equation (6). To do this, the value of the coefficient of variation corresponding to the system of batching, and of control generally, which it is proposed to

use must be known. Values have been suggested for the ratio $x_0/\bar{x}$ ($= 1 - 0.0233v$) to be expected with various levels of control. For example, in reference 3 a Table is given covering three levels of control. This is reproduced in Table 3, to which has been added a column giving the coefficient of variation.

TABLE 3.—CONTROL FACTOR AND COEFFICIENT OF VARIATION FOR THREE LEVELS OF CONTROL

(Reproduced from Road Note 4)

Conditions	Control factor, $c = \frac{100x_0}{\bar{x}}$	C.V.
Very good control with weigh-batching, moisture determinations on aggregates, etc.; constant supervision . . .	75	10.8
Fair control with weigh-batching	60	17.2
Poor control: volume-batching of aggregates	40	25.8

The procedure is to decide which level of control will be used, and then apply the control factor c to the specified "minimum" strength.

$$\text{Design strength} = \bar{x} = 100x_0/c \quad . \quad . \quad . \quad (7)$$

The validity of this procedure depends on the truth of the assumption that the coefficient of variation is the same, whatever the mean strength. As has been shown above, it is much more likely that the standard deviation is the same whatever the mean strength. This leads to a different method which is described below, and which it is suggested should be used, at any rate until a much fuller analysis of experimental data has been undertaken. There is no doubt that the proposed method is much nearer the truth than the current method.

Proposed Method

The proposed method, in which the standard deviation is taken to be independent of the mean strength, is even simpler. From equation (5):

$$\bar{x} - x_0 = 2.33s \quad . \quad . \quad . \quad . \quad (8)$$

Thus, given s , $2.33s$ is added to the "minimum" strength, giving the mean strength for which the concrete must be designed. All that is now necessary is to provide a Table giving $2.33s$ for various levels of control, corresponding to Table 3. Some data can be found in the literature on which to base the figures for a few levels of control, and a provisional Table is given in Table 4. In later sections are analysed the sources of variations contributing to the overall variation, and it is shown that the results of such an analysis are consistent with the values of the standard deviation used in Table 4.

TABLE 4.—STANDARD DEVIATIONS AND VALUES OF (MEAN—MINIMUM) STRENGTH FOR VARIOUS LEVELS OF CONTROL

Level of control	Standard deviation, s : lb./sq. in.	(Mean—minimum), $\bar{x} - x_0$: lb./sq. in.
Excellent . . .	400	930
Very good . . .	500	1,160
Good	600	1,400
Fair	800	1,860
Poor	1,000	2,330
" Uncontrolled " .	1,200	2,790

No attempt is made at present to describe in detail the methods necessary to achieve these levels of control. Briefly, however, it may be said that :—

- (a) "Excellent control" involves great care in proportioning, accurate estimation of the moisture content of the aggregates, or alternatively accurate control of the water content of the mix by controlling its workability, and very good supervision.
- (b) "Fair control" would apply to reasonably good volume batching, but without control of moisture content of the aggregates, the water addition being judged by the mixer operator. It represents a reasonable standard of work by the methods in use before deliberate control was introduced.
- (c) "Uncontrolled" refers to slapdash work in which mix proportions vary widely. It would not normally be found in civil engineering work of any importance.

Difference between Current and Proposed Methods

If the two methods are applied to concrete of one given strength they are obviously identical, provided the standard deviation used in the proposed method is, for the mean strength in question, equivalent to the coefficient of variation used in the current method. When they are applied to concretes of widely different strength, however, they inevitably lead to very different conclusions. As an illustration, let it be assumed that the three levels of control given in Table 3 are equivalent to the "very good," "fair," and "poor" of Table 4—this would make the methods agree approximately for concrete with a mean strength of 4,000 lb. per square inch. Table 5 gives the calculated mean strength by both methods for concrete with "minimum" strength ranging from 2,000 to 6,000 lb. per square inch.

The actual figures depend on the values taken for the standard deviation (S.D.) and coefficient of variation (C.V.), but the important feature, which does not depend on the particular values taken, is that the current method greatly exaggerates the mean strength required to achieve a high minimum,

TABLE 5.—MEAN STRENGTH CALCULATED BY CURRENT AND PROPOSED METHODS

Method of calculation		Current			Proposed		
Level of control		V.G.	Fair	Poor	V.G.	Fair	Poor
C.V. (per cent) or S.D. (lb. per sq. in.) .		10.8	17.2	25.8	500	800	1,000
		(1)	(2)	(3)	(4)	(5)	(6)
Minimum	2,000	2,670	3,330	5,000	3,160	3,860	4,330
strength :	3,000	4,000	5,000	7,500	4,160	4,860	5,330
lb. per sq. in.	4,000	5,330	6,670	10,000	5,160	5,860	6,330
	5,000	6,670	8,330	12,500	6,160	6,860	7,330
	6,000	8,000	10,000	15,000	7,160	7,860	8,330

particularly if control is not very good. Some of the strengths in columns (2) and (3) in Table 5 are not practicable, that is, they suggest that the given minimum could not be achieved except with "very good" control. The proposed method, however, suggests that they could be achieved with "fair" or even "poor" control. It is, of course, unlikely that any attempt to make very-high-strength concrete would be made without reasonable supervision, but the standard represented by "very good" control is a very high one, by no means common today, and unknown not many years ago.

The reason for the different results may be illustrated as follows: If concrete has a mean strength of 4,000 lb. per square inch, with "fair" control, the minimum strength of this concrete, according to the current method, is 60 per cent of 4,000 = 2,400 lb. per square inch, that is, 1,600 lb. per square inch below the mean. If the mean is increased by 1,600 lb. per square inch to 5,600, this does not raise the minimum to 4,000; the minimum is now 60 per cent of 5,600 = 3,360. The reason is that the coefficient of variation is assumed to remain constant, so that the standard deviation increases, when the mean is raised from 4,000 to 5,600; that is, it is assumed that, by increasing the mean, the variability is automatically increased. Therefore, the mean must be increased still further to account for this. If it is raised by 4,000—3,360 (= 640), it becomes 6,240, and the minimum is now 60 per cent of 6,240 = 3,744 lb. per square inch. This process of increasing the mean, first to allow for the difference between the specified and the actual minimum, then to allow for the consequent increase in variability, goes on indefinitely until a value of 6,670 lb. per square inch for the mean is reached asymptotically. In the proposed method, on the other hand, only the first step is required, since increasing the mean does not involve an increase in variability.

SUMMARY AND CONCLUSIONS: PART 1

The method at present used for designing concrete mixes to give a certain minimum strength has been examined, and it is concluded that the

method is based on a false assumption, namely, that the coefficient of variation is, other things being equal, independent of the average strength of the concrete ; in other words, the variability increases with the average strength. Available evidence strongly suggests that this is not so : the variability is the same, other things being equal, whatever the mean strength, over a wide range including all cases of practical importance. A design procedure based on this assumption is described. The two methods can lead to very different conclusions, especially when the minimum strength required is high.

Part 2 deals with the effect of the various factors influencing concrete strength, and an attempt is made to relate them to the observed variability.

PART 2

FACTORS AFFECTING CONCRETE STRENGTH

In order to achieve the desirable object of reducing variations in concrete strength, it is first necessary to identify the causes of variation, estimate their effects, and calculate the improvements to be expected if some of them are reduced or eliminated. Such an analysis has been attempted by several authors, but in general the methods used have been erroneous—particularly in the estimation of the combined effect of several causes of variation ; in view of the importance of the matter it is desirable to consider it in some detail.

If concrete is made from suitable materials, in suitable proportions, properly mixed, compacted, and cured, its strength, to a first approximation, depends only on :

- (1) the quality of the cement ;
- (2) the water/cement ratio (which will vary owing to batching errors) ;
- and (3) testing errors, that is, random variations in the measured strength of apparently identical pieces of concrete.

It is assumed, of course, that the mix used is capable of being compacted, and does not segregate.

These factors will be examined, and the extent of the variability resulting from them estimated. The water/cement ratio depends on the control of batching, etc. ; its variation is a complex function of several causes, and needs detailed analysis.

Testing Errors

By “ testing error ” is meant the variation between the strengths of cement cubes which are apparently identical (made from the same materials, with the same proportions, and, in particular, the same water/cement ratio) and which are mixed, compacted, and cured in as nearly as possible the same way. Such cubes do not give identical strengths, for reasons which cannot be identified, and the variation is denoted by “ testing

error.”* If two operatives prepare cubes made from “identical” mixes, and follow the same procedure throughout, the differences in strength include all sources of testing error. Cubes taken from the same batch of concrete, prepared by the same operative, cured in the same tank, etc., will show rather less variation, since some of the factors, such as the operative’s “personal error,” curing temperature, etc., are the same for all cubes. It is not easy to find or obtain data which enable a detailed analysis of the testing error to be carried out, but relevant literature contains data on which to base an estimate of overall testing error.

In a laboratory investigation Williams⁷ found a standard deviation of about 150 lb. per square inch for testing error, using 1 : 6 and 1 : 7 concretes. Data given by Graham and Martin⁶ have been analysed by statistical methods, leading to the following estimates :—

- (1) Small experiment on sampling and testing (see Appendix 1) :—

Testing error standard deviation = 150 lb. per sq. in.

Sampling „ „ „ = 57 „ „ „

The latter is so small that the true value may well be zero.

- (2) Larger set of results (see Appendix VIII of reference 6) :—

Testing error standard deviation = 266 lb. per sq. in.

This is derived from a figure of 188 lb. per square inch for the standard error of the mean of two results. If the tests were completely independent the standard deviation for a single result would be $188\sqrt{2} = 266$, but since they were not independent the true value is likely to be rather less.

- (3) Main body of data in Table XI of reference 6 ; these refer to site concrete :—

Testing error standard deviation = 304 lb. per sq. in.

It appears, therefore, that while a standard deviation of about 150 lb. per square inch is attainable in a well-equipped laboratory under ideal conditions, a figure of about 300 lb. per square inch should be used for work on site, even with a good standard of control.

Variations in Cement Quality

Various authors have recorded tests on several samples of cement, and have deduced from the results that cement variations are responsible for a large part of the variation in concrete strength. These deductions tend to be misleading, since the removal of the cement-quality variations would not reduce the variability of the concrete strength to anything like the

* This name is not a particularly exact one ; the variation does not all arise from the process of testing, and the word “error” does not mean an avoidable mistake. “Testing error” means the variation in test results arising from the process of sampling, cube-making, compacting, curing, and testing in the machine. It cannot be eliminated, although greater skill and care on the operator’s part may reduce it.

extent implied. It is, however, important to know what variations in cement-quality may be expected; the methods of assessing variability normally adopted are inadequate, and lead to serious over-estimates, since other sources of error are included. There is no fundamental difficulty in estimating the true variability, but because of other sources of variation it is necessary to plan and analyse the experiments correctly, and to work on a scale large enough for reasonably precise estimates to be made. The method usually adopted is to test each of a number of batches of cement, using say the British Standard test for compressive strength, and to assume that the variation observed among these batches is equal to the variation in cement quality, ignoring the effects of testing errors, etc. To illustrate the point, an analysis will be given of the data quoted by Graham and Martin ⁶ for a series of tests specifically designed to measure the effect of cement-quality variation.

The data consist of the strengths for a series of forty-two batches of concrete made during a period of continuous running under well controlled conditions. From each batch four test cubes were prepared; two were tested after 7 days and two after 28 days. Samples of cement and aggregate were taken from the materials used for each batch, and the moisture content of each of the three grades of aggregate was determined, so that the cement/water ratio for each batch was estimated as accurately as possible. From the samples of cement and dry sand, four test cubes were made according to B.S. 12; two were tested after 7 days and two after 28 days. The object was to assess cement quality from the B.S. (mortar) cubes (Graham and Martin call this the "cement" test) and compare the "cement variation" with the total variation in the concrete tests. The "cement variation," expressed as the total variation, or range, of the forty-two mortar tests, was found to be 2,595 lb. per square inch at 28 days.

The conclusion reached was that cement variations account for a variation in concrete strength of 2,100 lb. per square inch, equal to 48.2 per cent of the total variation in concrete strength, which was 4,350 lb. per square inch. The range of the mortar tests is 2,100 (7,480 maximum minus 4,885 minimum minus 500 for sampling and testing errors). As stated on p. 167 above, it is wrong to add the standard deviations (or ranges) for two separate sources of variation: they should be squared and added; the square root of the sum gives the combined standard deviation.

The experimental results can be analysed by statistical methods, leading to much more precise conclusions than can be reached by simple inspection; this is done below.

Effect of Cement/Water Ratio

The cement/water ratio ranged from 1.89 to 2.22, with a standard deviation of 0.0875. The consequent standard variation in concrete strength at 7 days is $0.0875 \times 2,570 = 225$ lb. per square inch, and at 28 days $0.0875 \times 3,700 = 324$ lb. per square inch. These account for the

greater part of the batching-error standard deviations, 299 and 462 lb. per square inch respectively.

Analysis of Variations in Concrete Strength

Variations in cement-quality must affect both mortar and concrete strength. Both are also affected by other sources of variation, but it is assumed that cement-quality is the only factor which affects both tests equally on every batch, that is, which makes them both give high or low results. Other sources of variation are assumed to be random, for example, testing error and batching error. If there were no batching errors in making the concrete, the concrete strength could be calculated from the mortar strength, and would differ from the calculated value only because of testing errors, but, owing to batching errors, the differences will tend to be greater than those arising from testing errors only. The extent of the difference between the calculated and observed values can be estimated by "Regression Analysis," in which the total variation of concrete strength over the forty-two batches is divided into two parts:—

- (1) Variation "due to regression" or the variation which can be accounted for by mortar strength variation, that is by cement-quality variations.
- (2) The remaining variation, which is attributed to batching and testing errors.

These calculations have been carried out for 7-day and 28-day strengths. Details are given in Appendix 2, and the main conclusions are as follows:—

- (a) The testing error standard deviation, for the mean of two tests, is 188 lb. per square inch. If these tests were entirely independent, the error standard deviation would be $188 \sqrt{2} = 266$ lb. per square inch. In fact they were not independent, and the true standard deviation may be rather less.
- (b) Cement-quality variation accounted for standard deviations of 227 lb. per square inch at 7 days and 253 lb. per square inch at 28 days. These cannot be regarded as different and a mean value of 240 may be taken. Taking $\pm 2.33 \times \text{S.D.}$ (the standard deviation) as the range which includes practically all results gives ± 560 lb. per square inch, or a total range of 1,120 lb. per square inch. This is considerably less than the figure of 2,100 given by Graham and Martin.⁶
- (c) Batching errors account for standard deviations of 299 at 7 days and 462 lb. per square inch at 28 days. If these are expressed as percentages of the mean strengths (that is, as coefficients of variation) they amount to 7.5 and 8.0 per cent respectively. This is an interesting confirmation of the deduction given on

- (d) The above three sources of variation operate independently, and to find the resulting standard deviation when any two or all of them operate the square root of the sum of the squares must be calculated. The following figures are the result:—

	7 days	28 days
Testing error (mean of 2 tests) :	188 lb./sq. in.	188 lb./sq. in.
Batching error :	299 "	462 "
Cement-quality variation :	240 "	240 "
Batching + testing :	343 "	499 "
Cement + testing :	305 "	305 "
Cement + batching + testing :	427 "	553 "

From the data given in reference 6 for routine concrete making can be found the following standard deviations.

	7 days	28 days
Testing error (single test):	304 lb./sq. in.	304 lb./sq. in.
Batch-to-batch variation; that is, cement + batching + testing (mean of two cubes) :	489 "	540 "
Cement + batching	439 "	495 "

These are in reasonable agreement with the previous figures, except that the testing error is somewhat greater.

If cement-quality variation were completely eliminated, the standard deviation would be reduced, at 7 days, from 427 to 343 lb. per square inch, that is, by 20 per cent, and at 28 days from 553 to 499 lb. per square inch, that is, by 10 per cent. There is thus little to be gained from reducing cement variation, contrary to the conclusion reached elsewhere. This statement applies only to the Heathrow data,⁶ and requires confirmation for cement obtained from a number of sources.

Relationship between 7-day and 28-day Strength

It may be noted that there is no evidence from these forty-two batches that there is any variation, from batch to batch, in the difference between 7- and 28-day strength. The average difference is 1,975 lb. per square inch (mortar test) or 1,759 (concrete test). The observed variations can well be accounted for by experimental error. The data do not support Graham and Martin's conclusion,⁶ based on the ratio of the strengths but taking no account of batching or testing errors, that the cements varied in rate of hardening. There must be some variation, but it is too small to be detected even in this quite large experiment.

Batching Errors

The making of controlled concrete is simply the technique of reducing batching errors, and the subject is, therefore, of great importance. Batching errors are the main source of concrete-strength variations, and are under the control of the site engineer. It would not be difficult to lay down arbitrary, but reasonable, values for the standard deviation to be associated with batching errors under different systems of control, but it is illuminating

where V is the variance, the square of the standard deviation. The reasoning is similar for the combined effect of several causes of variation.

It is required to estimate the variation in concrete strength resulting from variations in batch proportions. The main factor will be the cement/water ratio, but variations in aggregate/cement ratio entail variations in the cement/water ratio required for constant workability, so that they indirectly affect the strength. The batch can be controlled in one of two ways.

- A. All materials are carefully measured out, and the moisture contents of the aggregates are measured or estimated. The calculated amounts of cement, aggregates, and water are added, and no alteration is permitted (unless something has obviously gone wrong).
- B. The ingredients are measured out, perhaps with less care than in A above, but the water addition is at the discretion of the foreman or mixer attendant, who regulates it so as to give, in his estimation, constant workability. He may supplement his judgement by a test for workability, for example, a slump test.

Method B is associated with volume batching, and with concrete making generally when elaborate arrangements for weighing and moisture estimation are not in force. Method A applies only when such arrangements have been made, but the foreman may be allowed to use some discretion in adjusting the water addition. It would be expected that method A would result in better control over aggregate/cement ratio, but for control over cement/water ratio it is not clear *a priori* which method is the better. Although it is common to apply the term "controlled concrete" to concrete made by method A, control can be exercised by either method, and with weigh- or volume-batching.

It is now convenient to investigate the effects of errors in batching on concrete strength for methods A and B, with, in method A, no adjustment by the attendant. Method B is discussed on p. 189 *et seq.*

Method A : Weigh-Batching

The quantities C , A_2 , A_3 , W_1 , p_2 and p_3 , are all subject to error. Let their error variances (squared standard deviation) be :—

Weight of cement	.	.	.	V_1 lb. ²
Weight of sand	.	.	.	V_2 lb. ²
Weight of gravel	.	.	.	V_3 lb. ²
Weight of added water	.	.	.	V_4 lb. ²
Proportion of water in sand	.	.	.	V_5
Proportion of water in gravel	.	.	.	V_6

When all six quantities vary, the resulting variation in the cement/water

ratio, r , is given by the following formula, which is an extension of equation 10, p. 183 (see reference 4, chapter 3) :

$$V(r) = V_1 (\delta r / \delta C)^2 + V_2 (\delta r / \delta A_2)^2 + V_3 (\delta r / \delta A_3)^2 \\ + V_4 (\delta r / \delta W_1)^2 + V_5 (\delta r / \delta p_2)^2 + V_6 (\delta r / \delta p_3)^2 \quad (11)$$

The differentials are found from

$$r = C / (W_1 + A_2 p_2 + A_3 p_3)$$

and are evaluated numerically by substituting the nominal or average values of C p_3 . For 28-day strength :

$$x = 3,690r - 2,180$$

$$\therefore \delta x / \delta r = 3,690, \text{ or say } 3,700$$

$$\therefore s_x(x) = 3,700 s(r)$$

This is the standard deviation of concrete strength consequent on variations in r only.

When the differentials are evaluated and inserted in equation (11), it is found that :—

$$V(r) = 10^{-4} (2.21 V_1 + 0.0615 V_2 + 55.4 \times 10^{-4} V_3 \\ + 61.5 V_4 + 38.07 \times 10^4 V_5 + 131.10 \times 10^4 V_6) \quad (12)$$

Values of V_1 to V_6 for given systems of control are now required. The data available to the Author were rather limited, but measurements made under practical conditions on a few sites enabled reasonable estimates to be made. These were supplemented by information given by the Road Research Laboratory.² Fuller information from a large number of sites would be of great assistance.

Weight of Cement

If the cement is added in whole sacks, the weight will be fairly uniform. Observations suggest that a standard deviation of 1 lb. can be achieved, but that a much lower figure is unlikely. Considerably higher figures are not unknown. If the cement is added in half-sacks the variation will be much greater, say a standard deviation of 5 lb. or more. For bulk weighing on site the standard deviation may be as low as 1 lb. but will usually be greater, say about 2 lb., but with inadequate maintenance might be considerably greater.

Weight of Aggregates

For weigh-batching, or for carefully controlled volume-batching, with sand-bulking allowance, a standard deviation of 5 lb. can be attained. If the weigh-batcher is not well maintained, the deviation can rise to 10 lb. or more. For rather careless volume-batching 20 lb. or even more, is quite possible.

Weight of Water

If the weights of aggregates and the moisture contents are carefully controlled, it will be possible to measure out the required amount of water

very accurately, with a standard deviation of about $\frac{1}{4}$ lb. ($\frac{1}{5}$ pint). If control of aggregates is poor, it will be necessary, if a steady cement/water ratio is required, to adjust the water addition according to workability; the variation in total water content depends on the operator's skill and may be very large if supervision is poor.

Moisture in Sand

The moisture content of sand may vary widely, and large variations will upset both the aggregate/cement ratio and the cement/water ratio. Assuming that variations are reasonably small and do not seriously affect the aggregate/cement ratio, the figures adopted are 0.01 for good control and 0.02 for less careful control. In the case of the former, even if the moisture content varies by more than 1 per cent (as it usually does) it can be estimated closely enough for the uncertainty to be represented by a standard deviation of 0.01. The mean is taken as 10 per cent, so that in the two cases the ranges of variation are about 8–12 per cent and 6–14 per cent respectively.

Moisture in Gravel

The moisture content of gravel is assumed to have an average value of 3 per cent with standard deviations for two levels of control of 0.0025 and 0.005, giving ranges of variation of about 2.5–3.5 per cent and 2–4 per cent.

Calculated Variation in Strength due to Batching Errors

Appropriate values of the variances are now substituted in equation (12), taking three degrees of control which will be seen to correspond roughly to "excellent," "good," and "poor" as in Table 4. The standard deviations and variances are shown in Table 6.

TABLE 6.—STANDARD DEVIATIONS AND VARIANCES FOR THREE DEGREES OF CONTROL

	Standard deviation			Variance		
	Excellent	Good	Poor	Excellent	Good	Poor
Weight of cement	1 lb.	2 lb.	11 lb.	1	4	121
" sand	5 lb.	10 lb.	22 lb.	25	100	484
" gravel	5 lb.	10 lb.	22 lb.	25	100	484
" water	0.25 lb.	1 lb.	7 lb.	0.0625	1	49
Moisture in sand	0.01	0.02	—	10^{-4}	4×10^{-4}	—
" " gravel	0.0025	0.005	—	0.0625×10^{-4}	0.25×10^{-4}	—

The figures for "poor" control are based on ranges of variation given by the Road Research Laboratory.² In this case, a range is given for total water in the mix, the added water being roughly adjusted to give the desired workability. The figure of 7 lb. is therefore used as the standard

deviation of total water ; the variation in moisture content of the aggregate does not, therefore, arise—it is indirectly allowed for by the large standard deviation for total water in the mix.

Table 7 gives the values of $V(r)$, $s(r)$, and $s_r(x)$ found by substituting in equation (12). $s_r(x)$ is the standard deviation which would be found if variation or cement/water ratio were the only factor causing variation in strength. The overall standard deviation, allowing for cement-quality (S.D. = 250 lb. per square inch) and testing error (S.D. = 300 lb. per square inch) with three test cubes per batch is given by :

$$s^2(x) = 300^2/3 + 250^2 + s_r^2(x) \quad . \quad . \quad . \quad (13)$$

$s(x)$ is given in Table 7.

$V(r)$ is shown as the sum of the six terms of equation (11), so that the relative importance of the terms may be seen. For "poor control" the fourth term corresponds to the last three terms of equation (11).

TABLE 7.—CALCULATED VARIATION IN CONCRETE STRENGTH WITH DIFFERENT DEGREES OF CONTROL

Control	Excellent	Good	Poor
$V(r) \times 10^4$	2.21 + 1.54 + 0.14 + 0.38 + 38.07 + 8.19 = 50.53	8.84 + 6.15 + 0.55 + 6.15 + 152.28 + 32.78 = 206.75	267.41 + 29.77 + 2.68 + 301.35 = 601.21
$s(r)$ lb./sq. in.	0.0711	0.144	0.245
$s_r(x)$ "	263	533	906
$s(x)$ "	402	614	956

In the first two cases the dominant terms are those corresponding to variations in the moisture content of the aggregates. Unless these are much more accurately controlled, the other factors scarcely affect the result.

The values obtained for the overall variations correspond to those given in Table 4 in Part 1 of the Paper for "excellent," "good," and "poor" control. The Road Research Laboratory has stated³ that variations in water/cement ratio even larger than those used in Table 6 are possible, though not common. It seems reasonable, therefore, to take 1,000 lb. per square inch to represent "poor" control, and a higher figure, 1,200 lb. per square inch for virtually no control. It is interesting to note that the standard deviations in Table 4, which are reasonable in the light of much practical experience, can be derived from estimates of the contributory variations which are also in accord with experience.

VARIATIONS IN AGGREGATE/CEMENT RATIO

Variations in batch proportions also affect the aggregate/cement ratio. If some control over workability is exercised, the water added will be

adjusted so as to compensate so far as possible for the variations in aggregate/cement ratio, denoted by y . Ideally, whatever the value of y , the mixer attendant would adjust the cement/water ratio, r , to give the same workability each time, but in practice the actual value of r will not be exactly right. The effect on strength variation of exact adjustment can be calculated and a rough estimate of the effect of imperfect adjustment can be made. It is first necessary to calculate the variation in y for the three cases considered on p. 186, namely, "excellent," "good," and "poor" control.

The aggregate/cement ratio is given by:—

$$y = \frac{(A_2' + A_3')}{C}$$

$$= \frac{A_2(1 - p_2) + A_3(1 - p_3)}{C} \quad \dots \dots \dots (14)$$

$$V(y) = (\delta y / \delta C)^2 V(C) + (\delta y / \delta A_2)^2 V(A_2) + (\delta y / \delta A_3)^2 V(A_3) \\ + (\delta y / \delta p_2)^2 V(p_2) + (\delta y / \delta p_3)^2 V(p_3) \quad \dots \dots \dots (15)$$

When the numerical values of the differentials are substituted in equation (11) it becomes:

$$V(y) = 10^{-4} (28.70 V_1 + 0.646 V_2 + 0.750 V_3 \\ + 4.94 \times 10^4 V_5 + 17.01 \times 10^4 V_6) \quad \dots \dots (16)$$

The standard deviation of y for "excellent," "good," and "poor" control of batching is given in Table 8. As before, $V(y)$ is given in expanded form, to show the relative importance of the five terms. Also quoted are the "1 per cent limits" for y , that is to say, the limits within which nearly all values will lie (about 98 per cent).

TABLE 8.—VARIATION OF AGGREGATE/CEMENT RATIO WITH DIFFERENT DEGREES OF CONTROL

Workability medium; Nominal A/C ratio = 6

Control	Excellent	Good	Poor
$V(y) \times 10^4$	28.70 + 16.15 + 18.75 + 4.94 + 1.06 = 69.60	114.8 + 64.6 + 75.0 + 19.8 + 4.2 = 278.4	3,472.7 + 312.7 + 363.0 + 19.8 + 4.2 = 4,172.4
$s(y)$	0.0834	0.167	0.646
1 per cent limits for y	5.81—6.19	5.61—6.39	4.50—7.50

If the cement/water ratio were kept exactly constant, an aggregate/cement ratio of 6.8 would give low workability; 5.4 would give high workability. With "excellent" or "good" control, no batches would attain either high or low workability, and it would not be necessary to alter the

cement/water ratio because of changes in workability. With "poor" control of batching, however, batches of low or high workability would be common, and changes in cement/water ratio would be required. It is likely, however, that when control is "poor," no adjustments would be made, unless to add extra water to batches of low workability.

Method B: Adjustment of Water-Addition to give Constant Workability

It is usual, especially when control of batching and of the moisture content of the aggregates is not strict, for the mixer attendant or foreman to adjust the amount of water added so that, in his judgement, the workability of the mix is constant. If this method could be controlled in some way so that workability was very nearly constant, and if batching errors were kept small, a close control over concrete strength could be attained without the necessity of measuring the moisture content of the aggregates. Wide variations would upset the aggregate/cement ratio, but not usually so much that the consequent changes in cement/water ratio required to keep workability constant would cause serious variation in concrete strength. It is possible to calculate the variation in strength if it is assumed that the workability is kept absolutely constant, and it is possible to extend this to an approximate calculation for the case where the adjustment is not perfect.

Relationship between Aggregate/Cement Ratio and Cement/Water Ratio

It is necessary to know what adjustment of the cement/water ratio, r , is required to balance a change in the aggregate/cement ratio, y , in order to keep the workability constant. Table 5 in reference 3 gives, for four types of aggregate, four gradings, and four degrees of workability, corresponding values of water/cement ratio and y , for w/c ratios between 0.35 and 1.0. The relationship between r and y can be expressed in the form :

$$r = A + BY$$

$$\text{where } Y = \frac{1}{\sqrt{y+2}} \quad \dots \quad (17)$$

This purely empirical equation holds with satisfactory accuracy in all cases. The constants A and B vary; they have been estimated for all the sixty-four combinations in Table 5 of reference 3, and since B seemed nearly constant for a given aggregate type, whatever the grading number and workability, average values were calculated, as follows:—

Aggregate type	B
1. Rounded, $\frac{3}{4}$ inch	9.10
2. Irregular gravel, $\frac{3}{4}$ inch	11.06
3. Crushed rock, $\frac{3}{4}$ inch	11.21
4. Irregular gravel, $1\frac{1}{2}$ inch	9.58
Average	10.24

Using these values of B , average values of A were calculated ; these are shown in Appendix 3.

The measure of agreement with equation (17) is shown by the fact that, if r is calculated, the resulting values seldom differ by more than 0.07 from the values quoted in reference 3, 90 per cent are within ± 0.05 . This represents very satisfactory agreement—as good as can be expected when it is remembered that the aggregate/cement ratios are recorded to only one place of decimals. A difference of 0.1 in the aggregate/cement ratio means a change of about 0.04 in Y , and hence also in r . Thus alterations of 0.1 in the aggregate/cement ratio would remove practically all the discrepancies, and this is probably within the experimental error for the determination of aggregate/cement ratio.

From equation (17) and the values of $\delta x/\delta r$ given on p. 173 at 28 days it is found :

$$\frac{\delta r}{\delta y} = -\frac{1}{2} BY^2 ; \quad \frac{\delta x}{\delta y} = 1,850 BY^3 \quad . \quad . \quad . \quad (18)$$

Taking a mean value, 10, for B , and $y = 6$, as in the example on p. 183 :

$$\frac{\delta r}{\delta y} = -0.222 ; \quad \frac{\delta x}{\delta y} = -821$$

Thus an increase of 0.1 in the aggregate/cement ratio will require a decrease in cement/water ratio of 0.022 if workability is kept constant ; this leads to a reduction in 28-day strength of 82 lb. per square inch.

For “excellent,” “good,” and “poor” control of batching the following are the values of $s(r)$ and $s(x)$.

TABLE 9.—VARIATION IN CONCRETE STRENGTH : WORKABILITY CONSTANT

	$s(y)$	$s(r)$	$s(x)$: lb. per sq. in.
Excellent control .	0.0834	0.0185	68
Good control .	0.167	0.0370	137
Poor control .	0.644	0.1430	529

If, therefore, control of batching is good, and if the cement/water ratio is exactly adjusted to give the correct workability, the variations in strength resulting from variations in the aggregate/cement ratio are very small.

The Effect of Imperfect Adjustment of the Cement/Water Ratio

It is obvious that the cement/water ratio cannot be perfectly adjusted on every batch. Whether the workability is judged by the attendant, or estimated from a slump or other test, some variation must be allowed for. It will be assumed that if say, the desired workability is medium, the adjustment can be made sufficiently accurately to prevent workability

reaching more than half-way towards either high or low workability. However, a definition of the term "half-way" must be given.

In reference 9 the compacting factor, denoted here by F , is given for aggregate as :

Very low workability :	$F = 0.78$
Low workability :	$F = 0.85$
Medium workability :	$F = 0.92$
High workability :	$F = 0.95$

There is thus an increase of 0.7 between the classification limits of very low to low, and from low to medium, and of 0.3 between the limits of medium to high. The half-way points are 0.815, 0.885, and 0.935, and so the ranges for the four workabilities are taken :

Very low :	$F = 0.745 \text{ to } 0.815, \text{ or } 0.78 \pm 0.035$
Low :	$F = 0.815 \text{ to } 0.885, \text{ or } 0.85 \pm 0.035$
Medium :	$F = 0.885 \text{ to } 0.935, \text{ or } 0.91 \pm 0.025$
High :	$F = 0.935 \text{ to } 0.965, \text{ or } 0.95 \pm 0.015$

It may be assumed, then, that a skilled operator can usually maintain F within the range 0.91 ± 0.025 when aiming at medium workability, and take a standard deviation of $0.025/2 = 0.0125$; on the average, only one batch in twenty will be outside the range 0.885 to 0.935.

The variation of r thus entailed must also be known :

$$s(r) = s(F) \delta r / \delta F$$

Examination of the data given in reference 9 showed that, for a given mix, $\delta r / \delta F$ is constant; different aggregate types and aggregate/cement ratios give values ranging from -0.29 to -0.41 , with an average of -0.32 ; the latter figure is used, since the effect of the variation is negligible. The equation then becomes :

$$s(r) = 0.0125/0.32 = 0.040$$

This gives the error in r arising from failure to adjust the water addition exactly. It can be assumed that this error operates independently of the batching error, at any rate for not too wide variation in aggregate/cement ratio, so that :

$$V(r) = V(y) \left(\frac{\delta r}{\delta y} \right)^2 + V(F) \left(\frac{\delta r}{\delta F} \right)^2$$

which on substituting numerical values gives :

$$V(r) = 0.0493 V(y) + 16 \times 10^{-4}$$

For "excellent," "good," and "poor" control of batching this equation gives the figures shown in Table 10.

If, instead of assuming workability to be controlled to reach less than half-way towards high or low, it is assumed that it occasionally (once in twenty batches) reaches high or low workability, the values for $s(r)$ become

TABLE 10.—VARIATION IN CONCRETE STRENGTH : IMPERFECT ADJUSTMENT OF WORKABILITY

Control of batching	$V(y)$	$V(r)$	$s(r)$	$s(x)$ (lb/sq. in.)
Excellent . . .	70×10^{-4}	19.5×10^{-4}	0.0442	164
Good . . .	278×10^{-4}	29.7×10^{-4}	0.0545	202
Poor . . .	$4,148 \times 10^{-4}$	220.5×10^{-4}	0.1485	549

304, 326, and 606 lb. per square inch. Even with this latitude the effect on strength is small, and it is clear that control of workability can give an effective control of strength, even when the batch proportions are rather variable. The overall variations (including cement quality and testing error) corresponding to the cases tabulated above are :

Excellent :	S.D. = 345 lb. per square inch
Good :	S.D. = 363 " " "
Poor :	S.D. = 611 " " "

For this method of control, therefore, so far as strength variation is concerned, the first two are equivalent to "excellent," and the third to "good." It is of interest to note that with volume-batching of aggregates, under conditions in which the aggregate/cement ratio must have varied quite considerably, standard deviations of concrete strength below 400 lb. per square inch have been attained. It is clear from the above analysis that this is perfectly feasible, if the workability of the mix is maintained reasonably constant. There is a tendency to consider that elaborate arrangements for weighing materials accurately, and for estimating the moisture content of the aggregate accurately, are necessary to achieve consistent concrete workability and strength. This is evidently not so, but it seems likely that control may be more certain, and less liable to errors due to carelessness, if batch weights are carefully controlled. Some latitude must, however, be left to the attendant, unless the moisture content of the aggregate is very constant, or can be accurately determined beforehand.

In working through a heap of aggregate there may be sudden variations in moisture content, but generally, if reasonable care is used, changes will be gradual, so that a workability determination on one batch can be used to predict the approximate water quantity required for the next. This should assist in fixing at least an approximate value for the amount of water to be added, and enable better control to be achieved. Extreme care in measuring the aggregates, or in estimating their moisture content, is unnecessary. It is doubtless this feature which has enabled low variability to be achieved with volume-batching and no measurements of the moisture content of the aggregates, but with good supervision by the foreman or mixer attendant. On one site, in which the aggregates were

"measured" by a grab, a standard deviation of 400 lb. per square inch was observed in a set of one hundred tests taken over a period of several weeks. No system of weigh-batching and control could have given appreciably more uniform strength. If different concrete mixes were being made each day, however, this form of control might be less effective.

WEIGH- AND VOLUME-BATCHING

There is no doubt that volume-batching, carelessly done, leads to very wide variations in the aggregate/cement ratio and, unless the water addition is carefully controlled by the attendant to keep workability reasonably constant, in the cement/water ratio and hence in strength. Weigh-batching, if the batcher is well maintained, is less liable to wide variations, since the machine element tends to prevent the effects of gross carelessness, such as failure to fill gauge boxes, failure to tamp correctly, failure to allow for sand bulking, etc. If both methods are followed carefully, however, there seems no reason why weigh-batching should give more uniform strength or workability than volume-batching. It has been shown above that the effect of a wrong aggregate/cement ratio on strength is small, unless the error is very large—much larger than the best that can be achieved by careful volume-batching. Cases are known where well supervised volume-batching gave as good uniformity in strength as can be achieved by the most careful weigh-batching. This uniformity depends on the operator's skill in adjusting the water addition, but only in very favourable conditions can a similar, though perhaps smaller, adjustment be avoided in weigh-batching.

Weigh-batching offers advantages apart from uniformity of strength, but these need not be considered here. The association of the levels of control with weigh- or volume-batching has been avoided in this Paper, although it is probable that the low levels of control would in general be associated only with volume-batching, or with very unintelligently used weigh-batching.

The data used in this Paper are drawn from a limited number of sources. Without Graham and Martin's Paper,⁶ which gives the results obtained at London Airport in great detail, it would have been difficult to reach any worth-while conclusions about the effects of different sources of variation. The Author has criticized some of the conclusions reached by Graham and Martin, but the thoroughness of the collection and reporting of the test data are a model which could, with advantage, be followed in other construction jobs where concrete of different types, not necessarily so rigidly controlled, is being made. It may not be practicable to report lengthy series of results in technical journals, but these could be collected and filed by some central authority so that conclusions could be based on a truly representative collection of data.

SUMMARY AND CONCLUSIONS : PART 2

A study has been made of the factors which cause variation in concrete strength. These are divided into (a) cement quality, (b) cement/water ratio, and (c) testing error, which includes all factors resulting in variation among test cubes made from "identical" batches of concrete. The main conclusions are :—

- (1) Cement quality contributes much less to variability than has been supposed. On one very large job, cement quality accounted for only 10 per cent of the total variation in concrete strength. Faulty methods of deducing the effect of cement quality have previously led to much larger estimates.
- (2) Variations in cement/water ratio are the most important source of variation in concrete strength, unless they are very rigidly controlled. A thorough analysis is made of the factors causing variation in the cement/water ratio. It is shown that high accuracy in weighing materials is unnecessary, and does not ensure low variability, unless the moisture content of the aggregates can also be accurately measured. The most effective form of control is by workability, and if this is reasonably well controlled, very accurate proportioning is unnecessary. Good volume-batching is quite as satisfactory as weigh-batching, so far as uniformity of strength and workability are concerned.

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The Paper is accompanied by four sheets of diagrams, from which the Figures in the text have been prepared

APPENDIX 1

SAMPLING AND TESTING EXPERIMENT AT LONDON AIRPORT

The data given by Graham and Martin ⁶ are reproduced below in Table 11.

TABLE 11

Sample no.	Compressive strength at 28 days : lb. per square inch		
	Hand vibrated	Code of Practice	Average for sample
1	5,630	5,850	
	5,600	5,850	
	5,540	5,790	
	5,510	5,630	
Average . .	5,570	5,780	5,675
2	5,850	5,630	
	5,600	5,600	
	5,510	5,540	
	5,480	5,480	
Average . .	5,610	5,562	5,586
3	5,790	5,820	
	5,700	5,730	
	5,600	5,660	
	5,480	5,510	
Average . .	5,642	5,680	5,661
4	5,480	5,790	
	5,410	5,600	
	5,290	5,510	
	5,290	5,320	
Average . .	5,368	5,555	5,461
5	5,820	5,570	
	5,760	5,510	
	5,570	5,410	
	5,450	5,350	
Average . .	5,650	5,460	5,555
6	5,780	5,970	
	5,540	5,600	
	5,540	5,570	
	5,260	5,410	
Average . .	5,530	5,638	5,584
Grand average	5,562	5,612	5,587

Testing error is measured by the variation within a set of four results. Variation between the sample averages arises from sampling and testing errors.

The analysis of variance is shown in Table 12.

TABLE 12.—ANALYSIS OF VARIANCE

Source of variation	Degrees of freedom	Sum of squares	Mean square
Between methods .	1	31,000	31,000
Between samples .	5	240,850	48,170
Interaction .	5	230,150	46,030
Error . . .	36	805,400	22,370
Total . . .	47	1,307,400	

The factor "interaction" has the interpretation that if there is a real difference between the results obtained by the two methods, but this difference varies from sample to sample, there is said to be an "interaction" between the sources of variation, or factors, "methods" and "samples." In this case, there is no reason to expect such an effect, and since the mean square is not significantly greater than the error mean square, it can safely be assumed that there is no interaction.

The error mean square is an estimate of s_0 (s_0 denotes testing error standard deviation).

Thus $s_0 = 150$ lb. per square inch

The mean square "between samples" is an estimate of $s_0^2 + 8s_1^2$ where s_1 denotes the standard deviation arising from sampling error. It is not significantly greater than s_0^2 , that is s_1^2 might well be zero. The best estimate is:

$$s_1^2 = (48,170 - 22,370)/8 = 322.5$$

Therefore

$$s_1 = 57 \text{ lb. per square inch}$$

The sampling error, if it exists at all, is thus very small. The mean square "between methods" is a measure of the average difference between the results obtained by the two methods, plus testing error. It is not significantly greater than the testing error, that is, there is no evidence that the methods give different results. The average results by the two methods are 5,562 and 5,612 lb. per square inch; the difference, 50 lb. per square inch, is negligible; the true value may well be zero.

It could be that the two methods would give different testing errors, but in fact the estimates are 148 and 151 lb. per square inch respectively; there is no evidence that they differ.

APPENDIX 2

ANALYSIS OF VARIATIONS IN CONCRETE STRENGTH

Using the data given by Graham and Martin for mortar ("cement") and concrete strength at 7 and 28 days a regression analysis is first made (see reference 4, chapter 6) to find how much of the variation in concrete strength is accounted for by cement-quality variation, taking mortar strength as an index of cement quality, but allowing for the fact that the mortar test is subject to testing error. It is assumed that concrete strength is a linear function of mortar strength, that is, that:

$$C = a + bM \quad \dots \quad (19)$$

where a and b are constants and C and M denote concrete and mortar strengths respectively. This is a very reasonable assumption, at least over the small range of strength covered, and the data do not suggest that it is false. In regression analysis the total variation in concrete strength is analysed into two parts: (a) that corresponding to mortar strength variation, attributed to cement-quality variation; and (b) residual variation, attributed to batching and testing errors. If the graph of C against M gave points lying exactly on a straight line (equation 19), there would be no residual variation; in fact there is a considerable scatter.

Reference 6 gives results for forty-two batches of concrete, made under uniform

conditions, and these results are analysed below. The best-fitting straight lines relating C to M are :

$$\text{At 7 days : } C = 1,654 + 0.54 M$$

$$\text{At 28 days : } C = 2,629 + 0.50 M$$

The difference between the two values of b is not statistically significant, and a common value of 0.52 may be taken, giving :

$$\text{At 7 days : } C = 1,752 + 0.52 M$$

$$\text{At 28 days : } C = 2,485 + 0.52 M$$

These equations are valid only for the range of strengths covered by the data, and must not be used to extrapolate far beyond this range—as is clear if $M = 0$ is substituted.

The regression analysis is shown in Table 13. All results were rounded to the nearest 10 lb. per square inch to lighten the arithmetic.

TABLE 13.—REGRESSION ANALYSIS : CONCRETE STRENGTH

Source of variation	Degrees of freedom	7 days		28 days	
		Sum of squares	Mean square	Sum of squares	Mean square
Cement quality	1	2,284,900	2,284,900	2,942,200	2,942,200
Batching and testing	40	4,995,100	124,900	9,936,400	248,400
Total	41	7,280,000	177,600	12,878,600	314,100

The mean square for "cement quality" is much larger than that for "batching and testing," that is, concrete strength is undoubtedly correlated with mortar strength. The mean square for "batching and testing" is an estimate of $(s_0^2 + s_b^2)$, where s_0 denotes testing error standard deviation and s_b the batching error standard deviation (s_0 applies to the mean of two cubes). The mean square for "cement quality" is an estimate of $s_0^2 + s_b^2 + 42 s_c^2$, where s_c denotes the standard deviation for cement-quality variation. The factor 42 appears because forty-two batches were used.

The equations are therefore :—

		7 days	28 days
$s_0^2 + s_b^2 + 42 s_c^2 =$		2,284,900	2,942,200
$s_0^2 + s_b^2 =$		124,900	248,400
Therefore	$42 s_c^2 =$	2,160,000	2,693,800
	$s_c^2 =$	51,430	64,140
	$s_c =$	227 lb./sq. in.	253 lb./sq. in.

The difference between these values is not significant, and a common value of $\sqrt{(227^2 + 253^2)/2} = 240$ lb. per square inch is used for cement-quality variation, at both ages.

In order to deduce the value of s_b , it is necessary to know s_0 . This can be found from an analysis of variance for concrete strength. The total variation of concrete strength is divided into three parts :

- (1) Variation from batch to batch. This is attributable to batching errors and cement-quality variation.
- (2) The effect of age at time of testing (7 or 28 days).
- (3) Testing error. This is the variation which is not accounted for by (1) and (2).

It should be noted that (1) is based on the average of the 7 and 28 days' strengths.

The value of s_b derived from it will, therefore, be an average figure, since the batching error increases with increasing age.

The analysis of variance is shown in Table 14.

TABLE 14.—ANALYSIS OF VARIANCE OF CONCRETE STRENGTH.

Source of variation	Degrees of freedom	Sum of squares	Mean square
Between batches . . .	41	18,710,600	456,300
Between ages . . .	1	64,996,800	64,996,800
Testing error . . .	41	1,448,100	35,320
Total . . .	83	85,155,500	

$$\begin{aligned}
 s_0^2 &= 35,320 & s_0 &= 188 \text{ lb. per square inch} \\
 s_0^2 + 2s_b^2 &= 456,300 \\
 s_b^2 &= 420,980/2 = 210,490 \\
 s_b &= 459 \text{ lb. per square inch}
 \end{aligned}$$

Individual values of s_b for 7 and 28 days can also be derived. As shown on p. 198:

		7 days	28 days
$s_0^2 + s_b^2$	=	124,900	248,400
s_0^2	=	35,320	35,320
Therefore s_b^2	=	89,580	213,080
Therefore s_b	=	299 lb. per sq. in.	462 lb. per sq. in.

This completes the analysis of concrete strength variation.

APPENDIX 3

For a given type and grading of aggregate and a given workability, the relationship between y = aggregate/cement ratio and r = cement/water ratio is found empirically to be very closely:

$$r = A + BY \quad (20)$$

where $Y = 1/\sqrt{y+2}$

B is nearly constant, for a given aggregate type, whatever the grading number or workability. This covers the types and gradings given in reference 3.

The values of B are:

Aggregate type	B
(1) $\frac{3}{4}$ -inch rounded:	9.1
(2) $\frac{3}{4}$ -inch irregular gravel:	11.1
(3) $\frac{3}{4}$ -inch crushed rock:	11.2
(4) $1\frac{1}{2}$ -inch irregular gravel:	9.6

The value of A varies with aggregate type, aggregate grading, and workability. The values for the sixty-four combinations of these variables from Tables 5 and 6 in reference 3, are given in Table 15.

TABLE 15.—VALUES OF $(-A)$, THE CONSTANT IN EQUATION (20)

Workability	Grading number *	Aggregate type *			
		1	2	3	4
Very low	1	0.66	1.68	1.93	1.06
	2	0.69	1.74	2.04	1.07
	3	0.91	1.80	2.17	1.20
	4	1.06	1.98	2.24	1.32
Low	1	0.87	2.06	2.24	1.26
	2	0.92	2.05	2.29	1.27
	3	1.05	2.09	2.41	1.34
	4	1.22	2.19	2.47	1.46
Medium	1	1.15	2.30	2.49	1.51
	2	1.18	2.26	2.50	1.48
	3	1.23	2.36	2.57	1.52
	4	1.33	2.35	2.62	1.61
High	1	1.32	2.42	2.58	1.62
	2	1.30	2.36	2.57	1.59
	3	1.39	2.38	2.67	1.62
	4	1.50	2.47	2.72	1.70

* See reference 3.

Paper No. 5958

**“Guildhall : Provision of New Foundations to the North
Wall by Underpinning and Other Means”**

by

Burnard Geen, M.I.C.E.

(Ordered by the Council to be published with written discussion.) †

SYNOPSIS

Following the destruction of the roof and adjoining buildings in 1940, a temporary building was erected in 1951. During its construction disturbing facts were discovered and a report drew attention to a number of unusual and disquieting circumstances and to others which might occur, and recommended the provision of new foundations.

It was decided to support the wall on piles carried down to the blue clay and a novel means was adopted to support the buttresses.

By building a reinforced-concrete beam on each side against each buttress and over the piles and introducing stressing cables the buttress was squeezed as in a vice and the load transferred to the piles, thus avoiding any serious interference with the buttress or the soil below.

To prove the method the first buttress was tested by jacks.

All the difficult operations are described.

INTRODUCTION

THE construction of Guildhall in the City of London started in 1411 and is recorded as being uncompleted in 1439. The building is of great historic importance.

The Hall is 152 feet long and 50 feet wide, and prior to 1666, to judge from what remains, it had a Crypt under the main floor for its entire length. After the Great Fire of London, the roof and western half of the Crypt were destroyed, and today the eastern length of open Crypt is only 76 feet.

Since the construction of the Hall, the roof has been destroyed three times. The last occasion was in an air-raid on the City in 1940, when the arched hammer-beam roof was destroyed by fire and a great deal of destruction was caused to buildings close to the north side of the Hall.

In March 1951, a temporary building was being added on the north side to provide a meal service to the Hall, and certain excavations which

† Correspondence on this Paper should be received at the Institution by the 15th July, 1954, and will be published in Part I of the Proceedings. Contributions should be limited to about 1,200 words.—SEC. I.C.E.

were then made adjacent to the buttresses disclosed disturbing facts. A report on the matter submitted to the City Corporation, in November 1951, drew attention to these facts.

In studying the conditions of the structure, consideration had to be given to its ability to carry either a heavy timber roof exerting thrust, or a still heavier type of construction, based upon stone ribs of arch form; the choice was not made until nearly the end of the operations described below.

SITE EXPLORATION

A number of borings and trial holes were put down in the angles formed by the buttresses and the main wall, both on the north and south sides of the Hall. These showed that the foundation level varied throughout, in some cases ending in soft yellow clay above the main bed of ballast, which was only very little lower, and in other cases finishing in the ballast.

The wall panels differ in thickness and so the intermediate buttresses are not all the same size. The underside of the wall is not always the same on both sides at any section and varies in depth along its length. The buttresses vary in thickness, length, and depth, and even vary in depth in different parts of the same buttress.

It was ascertained that in numerous cases, though not all, the wall was at a much higher level than the base of the buttress and the buttress itself arched down from the wall to its base, which appeared to indicate that thrust had originally been catered for.

Another point which had to be borne in mind, in considering future building operations, was that heavy pumping might be required on the north side during Guildhall reconstruction work. It was always possible, owing to the high value of adjacent land, that future buildings with basements built in the vicinity might also involve heavy pumping, and possibly withdraw sand from the ballast and cause sinking of the walls and buttresses.

THE PROBLEM

A recommendation was made, therefore, that where the foundations of the north wall had been exposed to the elements for several years, owing to the destruction of adjacent buildings, and the loading conditions of the adjacent ground had been varied, it was advisable either to underpin the north wall, or otherwise provide satisfactory permanent foundations. In the latter case, it would be necessary to carry the foundations down through 18 feet of ballast, overlying 2 or 3 feet of stiff brown clay, to the blue clay below, which was fairly regular in depth at about 20 feet below Crypt floor level.

In March 1952, the recommendation that the foundations of the north

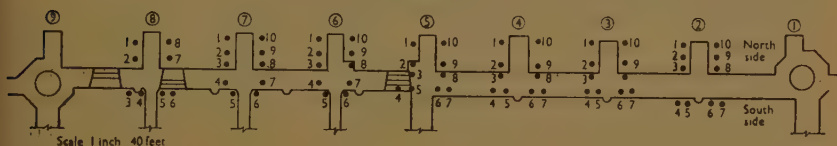
wall should be strengthened was approved, and a close and detailed consideration of the whole matter was put in hand.

In view of the existence of the temporary buildings when such operations were to be carried out, it was clear that the site conditions would be very difficult and that there would be little headroom to work in. In addition, access would have to be maintained for the service of the Hall until it was handed over to the contractors for reconditioning and construction of the new roof.

The construction of the lower part, at least, of the buttresses was very varied in character; some parts were of coursed stone, and others were formed by a conglomerate of stone, chalk, and flint, undoubtedly hand-picked and hand-placed.

It was observed that, whereas at the wall face the ground-water level was 6 feet 6 inches below Crypt floor level, it was 3 feet 9 inches lower 50 feet farther to the north.

Fig. 1



LAY-OUT OF PILES

Samples of the ground-water were tested to see if it would be likely to have a deleterious effect upon Portland cement, but these gave a negative result.

Compression tests on samples of the chalk, which formed the bulk of the lower portion of many of the buttresses, gave a minimum crushing strength of 56 tons per square foot.

It became evident that it was not possible to make any calculation regarding loads and ground pressures or to prepare a scheme until a complete survey had been made of the north wall. *Fig. 1* shows the proposed arrangement of piling at the buttresses.

Buttresses Nos 6 and 7 were, for all practical purposes, similar; all other buttresses varied, each requiring individual consideration and, to some extent, a different treatment. In one of the buttresses a large cavity was disclosed, and it was necessary to prove so far as possible the nature of the interior of the wall and buttresses. Eight holes were accordingly drilled into each buttress to see if cement could be injected under pressure and to enable an opinion to be formed as to the solidity or otherwise of the buttresses. In general, they were found to be very solid. This encouraged the opinion that, without undue risk it might be possible, with care, to cut away existing work and allow new work to be inserted,

UNDERPINNING

Although each buttress with its corresponding wall sections was treated on its individual merits, a method was devised which generally could be adapted to all of them. Broadly speaking, the seven buttresses fall into two groups: shallow buttresses, which terminate at or a little below Crypt floor level, and deep buttresses, which are founded 4 to 5 feet below Crypt level.

Of the seven buttresses which join the north wall (not counting the corners of the buildings), two were found to be deep, and five were shallow.

On the north face of the wall, that is, outside the building, it was permissible for the new construction to show, since it will be below basement level of the new Guildhall offices.

Consideration was given to various ideas, and it became clear that a normal method of stage-by-stage underpinning was unsuitable, as also was cementation, on account of the upper bed of clay.

Supporting by needles was also abandoned because of the friable character of the masonry, which in some of the buttresses might have involved the use of an excessive number of heavy needles; and it was not possible to avoid grave risk. In addition, since it was not known to what extent bond existed between the buttresses and walls, it was decided that another solution must be found.

The method adopted for supporting the buttresses consists (in all cases) of gripping them as in a vice, the jaws being formed by reinforced-concrete beams, 4 feet by 3 feet 3 inches in section, on either side, and the grip provided by three prestressing cables, each formed of eight 0.276-inch-diameter high-tensile steel wires. The cables are placed at one-third of the height up from the bottom of the beam, through 4-inch-diameter holes drilled through the buttress and corresponding holes left in the beams (*Figs 2 and 3*). The holes were subsequently grouted under pressure, after prestressing was completed. Each of the concrete beams is carried on three piles, which stand clear of the buttresses and take the loads down to the blue clay.

Thus, at no stage of the operation was the subsoil touched or the structure weakened, beyond the drilling of the three holes through the buttress to form ducts for the prestressing cables.

Buttress No. 2

It was decided to complete all operations for dealing with buttress No. 2 before commencing work on any of the others. It could thus be treated as a test case to establish the workability of the idea, further explore the fabric of buttress and walls, test the piles, and assess the time and cost of various operations with the object of preparing an overall programme for the six remaining buttresses. These could then be

Fig. 2



DRILLING BUTTRESS FOR CABLES

Fig. 3



TENSIONING OPERATION, BUTTRESS NO. 2

Fig. 5



WALL CUT AWAY FOR CRANKED BEAM, BUTTRESS NO. 2

Fig. 6



PILE TESTING

attacked simultaneously, subject to certain limitations dictated by the need for safety.

Piles and Beams (Figs 4)

From the detailed survey, the weight of the buttress, together with adjacent walls, was computed as 361 tons, based on a weight of 120 lb. per cubic foot of stonework. To this was added 62 tons as the estimated reaction from a stone-arch-type roof.

The type of pile was limited by site conditions, particularly headroom, and 19-inch-diameter bored in-situ piles of 45 tons nominal bearing capacity were chosen, and so arranged that the centroid of the pile group coincided as nearly as possible with the centre of gravity of the new superstructure.

This necessitated placing four of the ten piles required to cater for the total load of 423 tons, on the south side of the wall and in the Crypt.

Since it was not permissible for beams over these piles to show in the Crypt, the two beams which grip the buttress were continued through the walls and cranked down below Crypt floor level, where they are carried on two piles each. They act as ordinary needles so far as the wall portion of the superstructure is concerned. *Fig. 5* shows the space excavated to receive the cranked portion of a beam.

The dimensions of the beam were governed by the position of the piles, which were sunk as close as possible to the wall and buttress— $19\frac{1}{2}$ inches from face of wall to centre-line of pile—allowing about 9 inches from the side of the top lining tube.

This arrangement made the beams $2 \times 19\frac{1}{2}$ inches = 3 feet 3 inches wide, and the height of 4 feet was necessitated by consideration of shear strength as well as the use of this beam as part of a prestressed beam in its transverse direction.

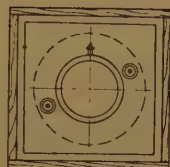
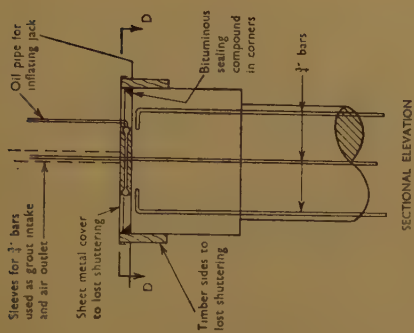
The lengths of piles were varied to give between 5 and 8 feet of penetration into the blue clay, and the final decision on depth was made on the basis of the loading-test results.

For this purpose—as well as to allow free movement of the beam during prestressing—separate caps were provided to the piles, with recesses over to allow the placing of hydraulic ship-jacks.

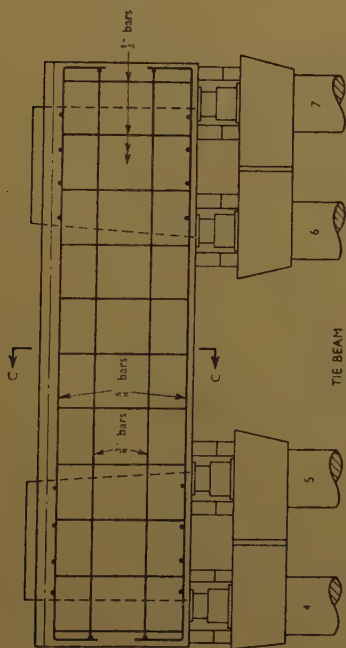
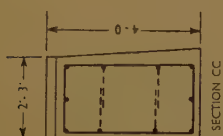
PRESTRESSING BUTTRESS

In order to assess the compressive force to be developed by the cables, the completed structure was considered as a prestressed beam formed of two concrete members over the piles and the intervening stone buttress, similar to the formation of a prestressed beam consisting of a number of prefabricated sections.

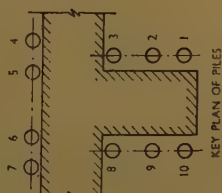
Since the compression is bound to spread through the buttress, a greater depth of beam can be assumed to be active in the centre of the



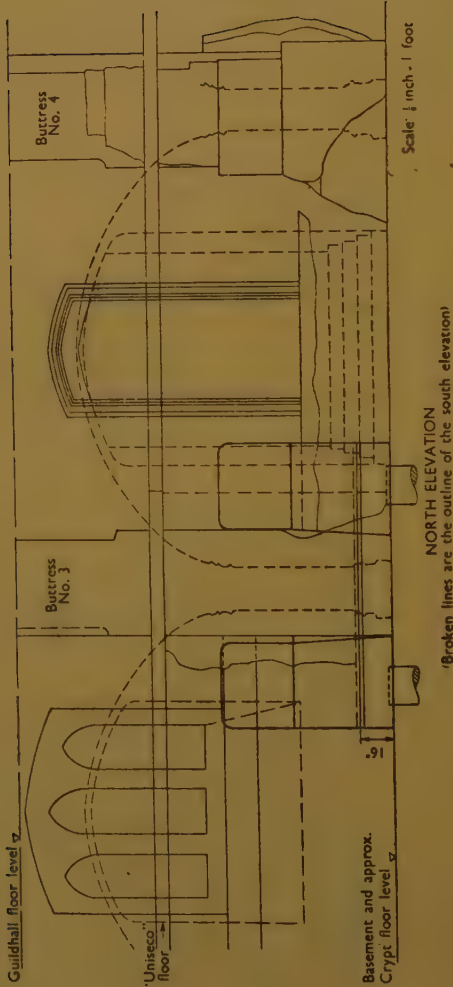
DETAILS OF FLAT JACK
Scale: $\frac{1}{4}$ inch = 1 foot

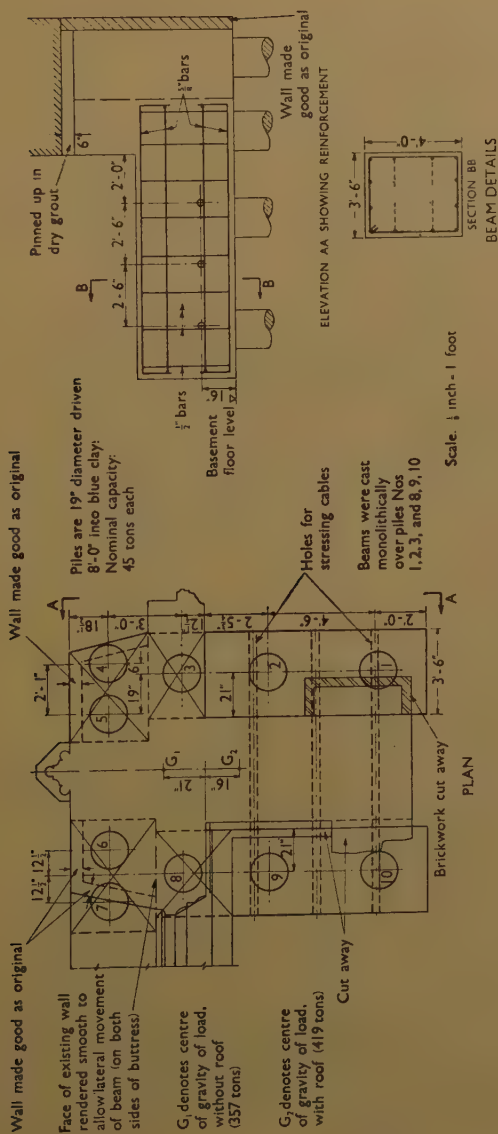


Concrete: Mix 1: $1\frac{1}{2}$: 3
Cover to bottom main bars $2\frac{1}{2}$ "
elsewhere 2"



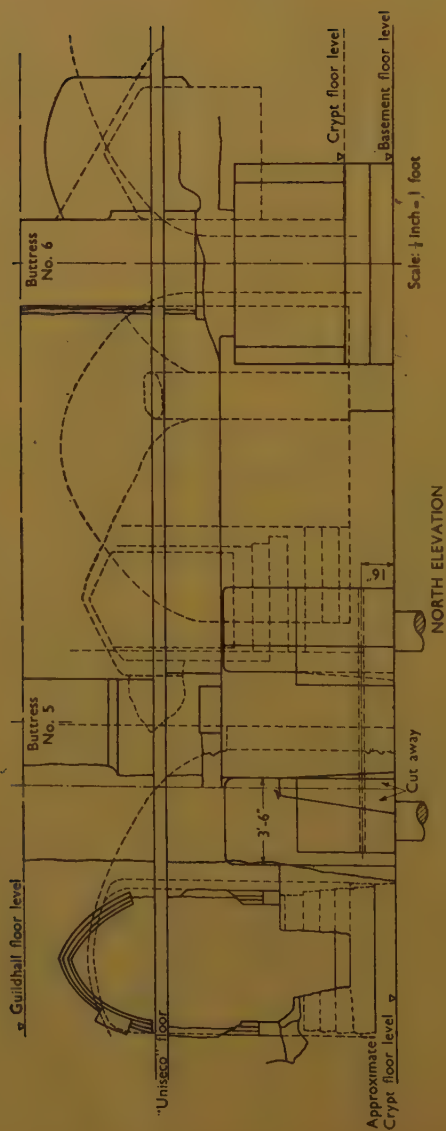
Figs 7

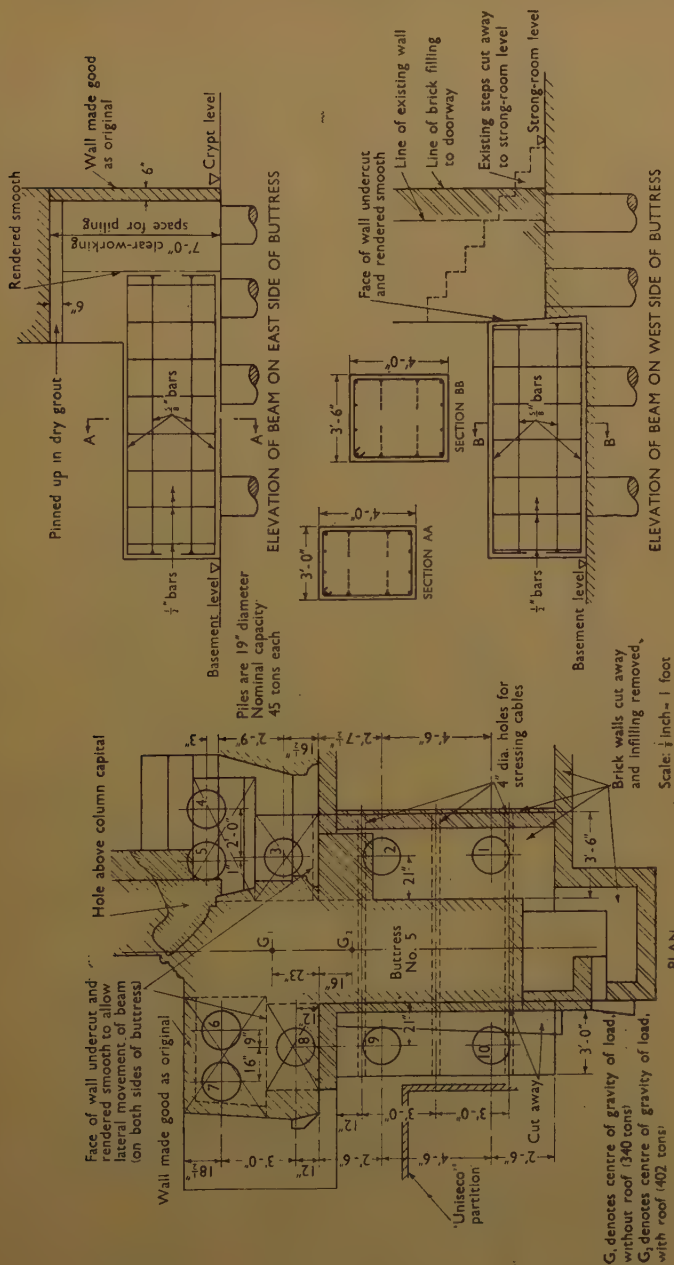




BUTTRESS No. 3

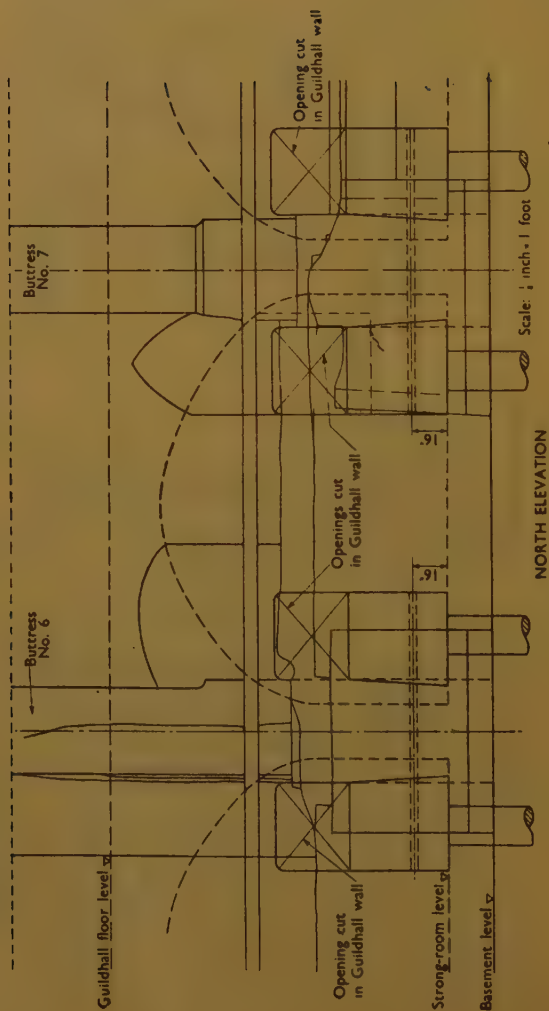
Figs 8

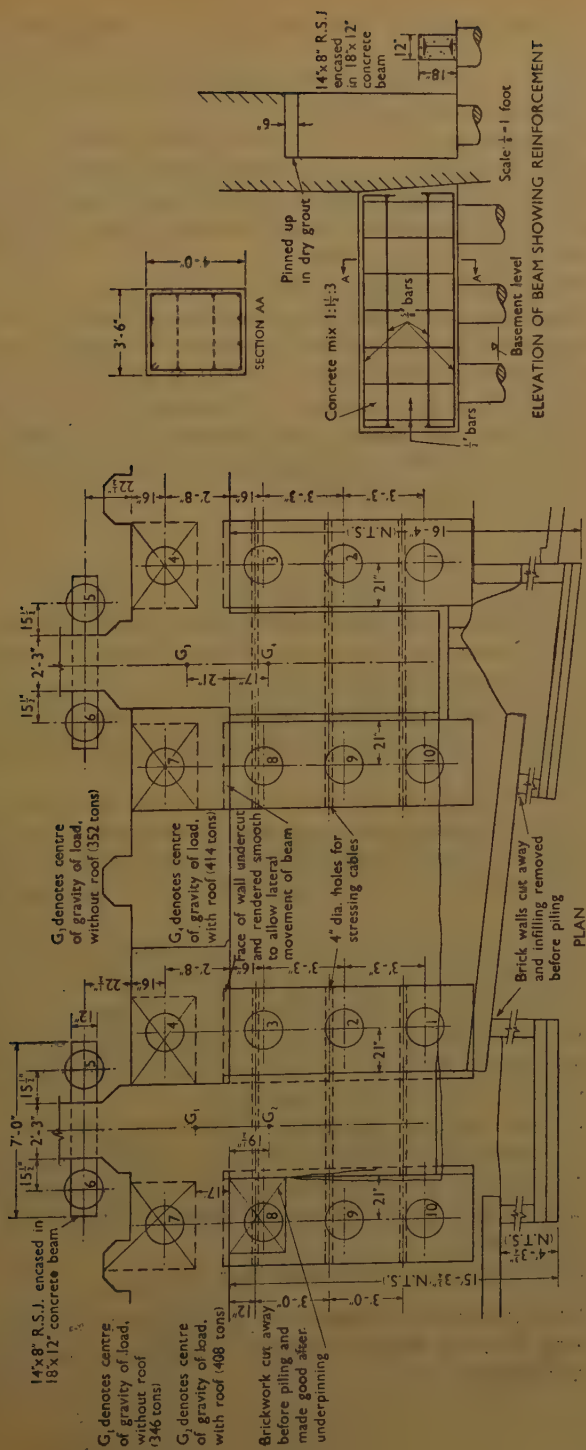




BUTRESS No. 5

Figs 9





BUTRESSES NOS 6 AND 7

buttress than on the 4-foot-faces of the concrete sections, and this was allowed at the moderate figure of 7 feet.

The stressing force per cable was fixed at 32 tons, being a computed 28 tons, plus an allowance of approximately 15 per cent for slip at the anchorage and creep; it does in fact produce a pressure almost the same as the pressure existing on the base of the buttress, so that no appreciable variation in stress would be introduced.

From the stress/strain diagrams of the tensile tests carried out on six samples of prestressing wires, the average modulus of elasticity was established as 30 multiplied by 10^6 , and the elongation necessary to produce a stressing force of 32 tons was calculated as 0.804 or $\frac{1}{16}$ inch.

The prestressing force was alternatively calculated as the tie counter-acting the thrust of an arch formed in the stonework of the buttress.

As an additional precaution, the buttress was slightly undercut where making contact with the concrete beams.

TESTING PILES

As part of the design, two of the reinforcement rods to the piles were carried up into the beam, running in sleeves to ensure free movement, and grouted after the prestressing had taken place. One of the rods was extended beyond the top of the beam, where it made contact with a Mercer gauge on which the movement of the piles when jacked down could be read.

Two 70-ton hydraulic ship-jacks were used. They were placed alternately over two corresponding piles, and a downward thrust of 50 tons was exerted on the piles. Prefabricated concrete wedges were inserted and driven home to stop the piles from rising after the test load was released (see *Fig. 6*).

In general, piles which penetrated 5 feet into the blue clay moved about $\frac{1}{8}$ inch or slightly more; of the two piles which penetrated 8 feet into the blue clay one moved 0.114 inch and the other 0.030 inch. Considering that these piles are 28 feet long, the latter figure is no more than the anticipated elastic shortening of the pile under load, and it may be assumed that the foot of the pile did not move at all.

In two cases, the provision of recesses for the large ship-jacks would have called for extensive excavation to allow access to them; consequently Freyssinet flat jacks were used instead and left in. The recesses formed for inflating these jacks were finally pressure-grouted through the sleeve of the recording pile bar.

The test results were satisfactory since they proved:—

- (a) That the length and character of the piles were satisfactory to carry the loads without a degree of movement likely to injure the structure.

- (b) That the friction developed between the beam sides and buttress faces was sufficient to ensure that the buttress could not force its way down between the beams.

It was then not necessary in the remaining beams to make the same provision for testing, but it was decided to bore all subsequent piles 10 feet into the blue clay.

SUMMARY

The novel method devised and adopted proved to have the following outstanding advantages :—

- (a) It was not necessary to excavate below the level of the existing foundations and disturb them.
- (b) It was not necessary during the process to weaken the buttresses seriously.
- (c) It permitted the supporting beams to be placed at various levels to suit the varying levels of the existing foundations.
- (d) When the operation on any buttress was completed, it became immediately effective.

In deciding the order in which the work should be done, consideration had to be given to the weakness developed at any time, and it was thought fit in the early stages of the work to deal only with alternate buttresses at one time.

Buttresses Nos 3 and 4.—These two buttresses, like buttress No. 2, are in line with the open Crypt and no construction work could be allowed to show on the Crypt side.

Before operations commenced in this case, detailed photographs were taken of the old wall, so that in the event of any cutting away being required new work could be matched with the old.

The operations to these two buttresses, which are similar in type, though not identical, consisted of the following. Piles Nos 1, 2, 9, and 10 were cast, then the wall was cut away to a height of 7 feet to allow pile No. 3 to be cast. The beam over piles Nos 1, 2, and 3 was added, and the wall built up tight against the head of the cavity (*Figs 7*).

A similar operation was then undertaken for pile No. 8, and the beam over piles Nos 8, 9, and 10 was added, and the cavity built up. The beams were then stressed to produce the required compression in the buttress.

The buttress having thus been made secure, the wall was then cut away for piles Nos 4 and 5, the piles were cast, and the wall was pinned up to within 6 inches of the old inner face to allow for new stonework, which was added later and carved to match the old work.

A similar operation was carried out for piles Nos 6 and 7.

With the very small working space and headroom, the piling was difficult and slow, but was accomplished without mishap.

Buttress No. 5.—This was similar in type to Nos 3 and 4 but slightly different in detail; on one side of the buttress an old window or door opening permitted the beam to be carried across piles Nos 8, 9, and 10, but on the other side the beam stopped short of pile No. 3 and only passed over Nos 1 and 2 (see *Figs 8*).

Cable stressing followed as previously described and the wall was then cut away for piles Nos 6 and 7 and pinned up before allowing the wall to be cut away for pile No. 8; this in turn was pinned up and refaced.

Buttresses Nos 6 and 7.—These were opposite one of the walls in the strong rooms (probably added after 1666), where the floor level is $20\frac{3}{4}$ inches above the basement floor level (see *Figs 9*).

Piles Nos 5 and 6 were first installed, and this allowed a needle to be inserted to carry part of the weight of the cross-wall. The main wall was then cut away for either No. 7 or No. 4 and after the pile was installed and pinned up the further hole was cut away for No. 4 or No. 7 and also pinned up.

Piles 1, 2, 3, 8, 9, and 10 were then installed, and the beams cast just into the main wall where a smooth joint was provided to allow the beams to move laterally when post-stressed.

After maturing the beams were stressed and grouted, and the whole foundation became alive.

Buttress No. 8.—This buttress, which carries less total load and supports a thinner wall, also came opposite a cross-wall in the strong rooms where the floor level was $20\frac{3}{4}$ inches above the level of the basement floor level. It was dealt with in a similar manner to buttresses Nos 6 and 7.

By placing two piles, provided with caps, in the strong rooms on each side of the crosswall, a needle could be inserted to carry part of the weight of the cross-wall and the buttress beams could run through the main wall of the Hall and transfer their proportion of load to the four strong room piles.

The buttress piles were first inserted, and the beam was cast (upon a bed of concrete $20\frac{3}{4}$ inches high) with a temporary stop until the cables had been stressed. The work was then completed.

Piling work was started with one rig, until the difficulties of the ensuing operations had been assessed. Certain pieces of special plant were prepared to suit the limited headroom, and a second rig was later brought into use.

In only one case was an underground obstruction met with and the boring put out of line. The hole was rebored, a grouting tube inserted, and the whole cavity filled with concrete and then injected under pressure. After maturing, the pile was tested and found to be satisfactory.

All beams were completed by the end of May 1953, and the last buttresses stressed by 19 June, 1953, at which date the north wall was supported by its new foundations and adjacent work under contract was free to be undertaken.

ACKNOWLEDGEMENTS

For the success of the operations much of the credit is due to the Contractors, Messrs Trollope and Colls Ltd, and to their sub-contractors the Cementation Co. Ltd.

The Author was the Consulting Engineer to the City Corporation, in association with Messrs W. S. Atkins and Partners; Mr Denis Shirman, A.M.I.C.E., rendered valuable assistance in the detail design and supervision of the work

Thanks are due to the City Corporation for permission to write this Paper.

The Paper is accompanied by four photographs and five sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

CORRESPONDENCE
on a Paper published in
Proceedings, Part I, May 1953

Paper No. 5915

" Entrained Air in Concrete " †

by

Peter Joseph Frederick Wright, B.Sc.

Correspondence

Mr E. W. W. Richards was of the opinion that the Author, in suggesting that air-entrainment was unlikely to prove an asset in the construction of concrete roads of high quality in Great Britain, and that it would only find a place in low-grade construction by inexperienced contractors or under inadequate supervision, might have understated the importance of the subject and have missed the essential point lying behind American development of that process. American concrete-road-making plant, developed for high productivity, required a high degree of workability for its most effective operation. It was a common, but mistaken, criticism of American practice to say that that was a defect in American technique, or to allege that American concrete research was directed to finding palliatives for faults arising out of incorrect methods. American engineers were as well aware as British engineers were of the effects of workability and water/cement ratio upon concrete strength. They were of the opinion, however, that productivity came first, and that to sacrifice it for the sake of saving a relatively small quantity of cement or of avoiding occasional trouble from other causes was not good economics.

Mr Richards recalled raising that specific point with a party of American engineers and being told " To understand this you must appreciate our position and what we are trying to do. We know that we could make stronger concrete, and avoid some of our difficulties, if we made a stiffer mix, as you do. But if we did this it would mean slowing down our rate of progress and re-designing our equipment. What we are trying to do is to build roads cheaply and quickly, and to get as many miles as possible into the service of the community as quickly as possible. We reckon to

† Proc. Instn Civ. Engrs, Part I, vol. 2, p. 337 (May 1953).

deal with any difficulties arising out of this as we go along." Judging from their present position it would seem that they were successfully doing so, and thus enjoying the best of both worlds.

As for the extent of the use of air-entrainment, he had been told at the United States Bureau of Reclamation Concrete Laboratory at Denver, Colorado—possibly the finest cement- and concrete-research station in the world—that it was almost universal (the air-entraining agent being generally incorporated in the cement, so that the question of an additional operation or closer control in concrete making did not arise) and that it would now be unusual to find concrete made in the United States in any other way.

Whilst appreciating its academic disadvantages (and the indeterminate nature of concrete-road design), the use of air-entrained concrete had much to offer the constructional engineer in the way of expeditious and satisfactory construction, and notably with centrally-mixed concrete, so commonly used in the United States. It would be unfortunate if its use were relegated in Great Britain to the comparatively minor purposes suggested in the Paper without those aspects being considered.

Mr R. G. Smith, of Vancouver, observed that the use of entrained air in concrete had been brought to his attention recently in the construction of a prestressed concrete bridge at Bernera, Scotland. In the initial stages of trial mixes for that job, a concrete mix with a cement to aggregate ratio of 1 : 5 by weight, using $\frac{3}{4}$ -inch-diameter irregular gravel aggregate and a water/cement ratio of 0.40 was tried. That gave "low" workability with a compacting factor of 0.78 and was found to be too harsh. The contractor had suggested the use of an air-entraining agent to increase the workability but it was found that that had little effect on the mix. Such a result was confirmed by the curve 1 : 6/0.45, in *Figs 1 (a)*.

It would appear that for mixes richer than 1 : 6, where low workability was necessary, the use of an air-entraining agent, although increasing workability slightly, would reduce the compressive strength by too great a proportion. It would be interesting to know whether the Author had carried out any tests on mixes richer than 1 : 6 with "low" workability.

In the United States it had been suggested that air-entrained concrete should be used for prestressed concrete construction, and at least one prestressed concrete bridge had been constructed in that manner. Since most concrete required for prestressed concrete work was of a relatively rich mix with low to medium workability, Mr Smith was of the opinion that it was difficult to justify the use of air-entrainment in prestressed concrete construction, unless of course the structure was to be subjected to severe frost action. The Author's views on that subject would be appreciated.

With regard to the mix design proposed by the Author (see p. 355), Stage (1) stated that "The mix is designed as for normal concrete, by any reliable method, such as that set out in Road Note No. 4."

It should be remembered, however, as the Author had pointed out.

that the use of an air-entraining agent constituted the addition of another variable factor in the control of concrete. That was bound to increase the coefficient of variation of concrete strengths, so that concrete with the addition of an air-entraining agent should be designed for a higher compressive strength than with plain concrete. It would, of course, be necessary to know what degree of variation of percentage air-entrainment could be expected on an actual job but, in Mr Smith's opinion, some allowance for that factor should be made in the design of the mix.

Mr M. F. Kennard, referring to the survey of the use of air-entraining agents in concrete, remarked that no mention was made of its use in connexion with concrete placed by means of a concrete-pump. The water/cement ratio of pumpable concrete was often about 0.70 or higher, and consequently the compressive strengths obtained were comparatively low, and so an increased cement content was often resorted to.

On one particular contract it had been found that the addition of an air-entraining agent had enabled a drier and leaner mix to be used with the same or higher compressive strengths as previously. Because of the increased volume of the mix, and the saving in cement, there was also a slight saving in the cost of materials per mix. Since the mixer driver added the agent into the rotating mix, there was no additional labour or plant cost involved. It was expected that the amount of entrained air might decrease during a long length of pumping, but tests taken, although not sufficient in number to be conclusive, did not show any such loss. The lower amount of air entrained at higher temperatures, mentioned on p. 351, had been noticed, and it could be sufficient to offset in part the saving mentioned above.

With normal concrete without an air-entraining agent, tests taken with an air-in-concrete meter of the type illustrated on p. 354, had shown an air content of about 1.5 per cent. Although it could be said that that concrete was incompletely compacted, Mr Kennard considered that most samples would show an air content when tested in a similar way. In comparing the two mixes on p. 349 did the Author mean that the 5 per cent of air is the total air or the entrained air. If it was the former, the increase in volume would be less than 5 per cent.

Mr P. L. Melville, of Virginia, observed that on two points, the Author's opinions were somewhat in conflict with current views of the Virginia Council of Highway Investigation and Research.* The Virginia Department of Highways was now using air-entrained concrete on all construction work because it was believed that with *ad hoc* controls there was hardly anything to lose (negligible loss in strength, low cost of air-entraining agent) and much to gain (increased workability, cohesion, and homogeneity, reduced bleeding, savings due to higher yield) through air-entrainment. It should be noted that increase in durability was not a

* A co-operative organization sponsored jointly by the Virginia Department of Highways and the University of Virginia, U.S.A.

paramount factor since, owing to economic reasons, most of the Departments' concrete (that was, all paving) was used near the eastern seaboard where climatic conditions were not usually very severe.

The Author had mentioned the possibility of "wet batching" by transporting air-entrained concrete in dump trucks, but did not appear to encourage such practice. In 1952, the Council had undertaken an investigation into the transit of air-entrained concrete in non-agitating trucks. From a small mixer, centrally located, concrete having a slump of 2 to 4½ inches and 4 to 6 per cent of entrained air was transported to a series of small bridges. The times of haul ranged between 6 and 26 minutes and the distance between 2.3 and 15.5 miles. As a result of that study the following was reported :

- (1) The trucking of air-entrained concrete was accompanied by a reduction of the slump and air content, caused possibly by compaction in transit.
- (2) The loss of air and slump was not a function of time of transit or the distance above certain minima (probably about 5 minutes and 2 miles).
- (3) As a direct result of (1) and (2) above the compressive strengths increased with the trucking.
- (4) The durability through cycles of freezing and thawing as evaluated by the sonic modulus of elasticity was not appreciably affected and was satisfactory in all cases.
- (5) There was no evidence of segregation.

From those results it was concluded that :

- (a) It was safe and reliable to use non-agitating trucks.
- (b) The procedure might be used for transit times up to about 30 minutes regardless of mileage.
- (c) The percentage of air should be maintained at or near the upper limit allowed by specifications.
- (d) Opportunity for moisture losses in transit should be minimized (for example, by the use of tarpaulin cover, good condition of dump-truck).

As a result, the procedure was now commonly used for maintenance or incidental construction at savings to the Department of Highways.

With reference to the control of the admixture, current specifications required that the agent be added at the mixer either to the mixing water or to the sand. Most contractors had installed an approved automatic dispenser. Its use was desirable, but certainly not essential. It was found that manual dispensing could be reliable : an inspector would place the admixture from a calibrated measure, usually a properly trimmed tin cup, although in at least one case an empty mineral-water bottle had been found to be the right-size container to obtain 4 to 6 per cent of air. No difficulties had thus been encountered in dispensing the admixture.

Mr D. D. Fraser, drawing attention to the fact that the North of Scotland Hydro-Electric Board was carrying out a full-scale test of air-entrained concrete on the Gaur hydro-electric project, Perthshire, explained that it had been felt that on the high exposed locations of the Board's many projects in the Scottish Highlands, concretes possessing increased resistance to frost attack were not undesirable. Lower permeability and greater resistance to chemical attack were also attractions in structures which would be in permanent contact with corrosive moorland waters.

Air-entrained concrete had been in use at Gaur for the past year. During that time, about 5,000 cubic yards of mass concrete and 5,000 cubic yards of reinforced concrete had been poured in different types of structures, using a normal range of mixes. Naturally, insufficient time had elapsed, as yet, to judge whether air-entrained concrete was likely to prove more durable than ordinary concrete under the conditions prevailing. As regards permeability, however, laboratory percolation tests had indicated that for pressures of up to 100 lb. per square inch, air-entrained concrete was 50 to 100 per cent better than ordinary concrete, although for pressures in excess of 100 lb. per square inch there appeared to be no appreciable improvement.

On the site, air contents of 3·5 to 4·5 per cent were entrained, the agent used being the triethanolamine salt of a sulphonated hydrocarbon referred to by the Author. For most mixes, to offset any possible reduction in strength, the sand content was cut by 100–125 lb., and the water content by 30–40 lb. (both per cubic yard of mixed concrete). The improvement in plasticity and workability was quite noticeable and was a constant attraction to the placing squads. Even for concretes of identical slump, it had been found that the air-entrained variety was more workable than the ordinary. There was also a lessening in the tendency of some mixes to segregate during transportation, and that was particularly noticeable at the main dam to which the 8·5 : 1 hearting concrete—having 2½-inch aggregate—had had to be hauled in lorries over a distance of ½ mile.

Regarding the combined effect on strength and workability, Table 3 gave the average results of field test cubes made from air-entrained concrete during the past year and from ordinary concrete during an immediately preceding four-months period. Because more cubes had been made during the air-entrained period than during the ordinary period, some of the results were not strictly comparable, but nevertheless they did give some indication of the practical effects of an air-entraining agent.

It would be seen that the air-entrained concrete had been more workable than the ordinary concrete, as evidenced by the slight general increase in slumps. On strength and density, the agent seemed to have had no decisive effect one way or the other; in some cases there had been gains, in others losses, but on the whole the variations had been small. The effect of the agent on the lean 8·5 : 1 mix was worth attention, however, since in that particular mix, improvements in strength, density, and

TABLE 3.—ANALYSIS OF FIELD TEST CUBES TAKEN FROM ORDINARY CONCRETE AND AIR-ENTRAINED CONCRETE USED ON THE GAUR HYDRO-ELECTRIC PROJECT

(Ordinary Portland cement used throughout; figures in brackets refer to number of cubes tested.)

Aggregate/cement ratio and type of concrete	Max. size of aggregate: inches	Average slump: inches	Required compressive strength at 7 days: lb. per sq. in.	Actual average compressive strength at 7 days: lb. per sq. in.	Average density of 7-day cubes: lb. per cu. ft.	Pequired compressive strength at 28 days: lb. per sq. in.	Actual average compressive strength at 28 days: lb. per sq. in.	Average density of 28-day cubes: lb. per cu. ft.
4 : 1 normal . . 4 : 1 air-entrained	$\frac{3}{4}$ $\frac{3}{4}$	1.8 (13) 2.7 (9)	2,700 2,700	4,430 (16) 4,550 (18)	151.0 (20) 150.0 (18)	4,000 4,000	6,460 (7) 6,220 (5)	151.5 (2) 150.5 (5)
5 : 1 normal . . 5 : 1 air-entrained	$\frac{3}{4}$ $\frac{3}{4}$	1.4 (20) 2.2 (61)	2,350 2,350	3,920 (14) 3,540 (84)	150.4 (20) 150.5 (20)	3,500 3,500	5,290 (7) 5,640 (44)	149.0 (8) 150.5 (40)
5 : 1 normal . . 5 : 1 air-entrained	1 $\frac{1}{2}$ 1 $\frac{1}{2}$	1.3 (16) 1.0 (16)	2,350 2,350	3,880 (32) 4,010 (32)	150.8 (32) 151.3 (32)	3,500 3,500	5,190 (16) 5,720 (16)	151.0 (7) 151.9 (16)
6 : 1 normal . . 6 : 1 air-entrained	1 $\frac{1}{2}$ 1 $\frac{1}{2}$	1.3 (15) 1.4 (6)	2,000 2,000	3,510 (25) 3,050 (14)	150.5 (20) 150.0 (13)	3,000 3,000	4,420 (10) 4,370 (6)	151.5 (4) 150.5 (4)
8.5 : 1 normal . . 8.5 : 1 air-entrained	2 $\frac{1}{2}$ 2 $\frac{1}{2}$	0.4 (15) 0.5 (16)	1,300 1,300	2,050 (28) 2,160 (28)	150.0 (16) 150.4 (20)	1,950 1,950	2,720 (11) 3,000 (10)	150.5 (8) 152.0 (9)

workability had all been obtained. That bore out the Author's remarks and was in agreement with site opinion that the agent had a more beneficial effect on the leaner harsher mixes using $1\frac{1}{2}$ -inch and $2\frac{1}{2}$ -inch aggregates than on the richer mixes using $\frac{3}{4}$ -inch aggregate. In the 8.5 : 1 mix, the aggregate was held in position much better, the workability improved to a greater extent than the tiny increase in slump might suggest, and placing and consolidation were made easier. Perhaps that was the explanation of the increased strength and density.

The Author had rightly pointed out that air-entrainment was not likely to be adopted generally in Great Britain. However, for large mass structures using lean mixes, such as dams, it appeared that it might prove advantageous. Nor were possible improvements in the durability of thin-walled structures, such as aqueducts, to be rejected lightly, especially when all those advantages were to be had at little or no extra cost.

Dr M. M. Kansoh, of Alexandria, referring to the statement on p. 340: "The improvement in workability has not been satisfactorily explained," observed that a successful foaming or air-entraining agent should contain a wetting or so-called surface-acting agent in addition to agents which improved the stability of the foam produced. A satisfactory explanation of the improvement in workability arose from the knowledge of the ability of surface-acting agents to keep fine particles in suspension, which facilitated the motion of individual sand and cement particles past one another.

It was stated on p. 346 that "an increase in air content results in a loss of strength, but the entrainment of air enables the water/cement ratio and the sand content to be reduced substantially, thereby regaining most if not all of the lost strength." Responsibility for regain of strength was also a known attribute of the wetting agents which assisted in the wetting-out of the dry cement particles, with the result that the amount of dry or imperfectly hydrated cement in the mixers greatly reduced or, in other words, the cement was fully or more efficiently utilized. Ordinary concrete containing no such agent could sometimes be re-set after being crushed and moistened. That was an indication of the existence of dry or imperfectly hydrated cement.

Reduction of water content resulted in a reduction in cracks during drying out. That was another advantage to be added to the credit of air-entrained concrete, especially in "precast concrete" industry where too-wet mixes were used (p. 338), and in the construction of liquid containers.

Regarding the measurement of air content, the gravimetric method was the most accurate one. The standardization of vibration methods did not seem to be impossible. At least, a vibrating table of chosen stroke and frequency could be used for a standardized length of time. Such a method could be used in the laboratory and, wherever possible, in the field to eliminate the human element. In other cases, results obtained

from one such apparatus could be used, as a measure, to standardize manual methods of vibration.

The volumetric method was doubtful because the air could not be completely removed by agitation since the air bubbles "are very stable" (p. 342). In any case, the volume of displacing water represented not only the volume of entrained air, but also the volume of the voids which existed unintentionally in the concrete.

The pressure method could give only relative, not absolute, results. The apparatus should perhaps be calibrated before use.

Could the Author give some account of the effect of air-entrainment on heat and sound conductivity and on the tensile and bond strength of concrete, all of which one would expect to be improved, the first two as a consequence of the existence of billions of air bubbles, and the last two on account of minimizing the number and depth of cracks?

Mr G. S. Eldridge, of Canada, remarked that air-entraining admixtures in concrete had been in use in Vancouver, B.C. since 1929. At that time, central mixing plants had just become general, and the matter of transportation of concrete without segregation had been a problem. Adding fines did not solve it. Certain waterprooferers had shown a marked improvement in transportation in fixed-body bathtub trucks, even over plain concrete mixes in revolving-drum trucks, and it had appeared that in that existed a principle of fundamental importance. It was soon found that the amount of air added to a mix was the criterion of non-segregation and of workability of the concrete. Too much air gave great stability to the mortar, but reduced the strength. The greater the percentage of air; the greater the reduction in percentage of water, but there was a definite point (between 3 and 4 per cent of air) where the loss of strength due to increase in air more than offset the benefit of the reduction in water.

What really had delayed the general acceptance of air-entrained concrete was the fact that 24 years previously, concrete mixes were designed on a maximum-density basis, and attempts were made even to remove (by vacuum) the 1 per cent of air usually contained in plain mixes. Those mixes were harsh, and were very difficult to place without gravel pockets, so that there was very little maximum density, or even water-proofness, in the emplaced concrete.

Generally speaking, plain concrete required about 10 per cent more water than did air-entrained mixes with about 4 per cent air and properly controlled sand and slump. MacMillan, of the Portland Cement Association, had stated in 1930 that the main problem remaining unsolved after 30 years of research was the elimination of the 10 per cent of *excess* water needed in plain mixes in order to get reasonable placement. It was that 10 per cent of excess water that caused most of the troubles with concrete through bleeding, forming water channels, and floating cement to the surface, resulting in laitance, and giving poor bond between pours in walls and dusting on slabs.

At the outset of his investigations, Mr Eldridge had felt that air-entrained concrete would not be accepted unless it produced equal strength for equal cement, and it was not long before he was able to achieve that. Since then, numerous tests had been made in many independent laboratories which proved that equal strength could be obtained from air-entrained concrete using equal cement. It was true that that was contrary to many theories published at the present time, but a recent pamphlet by the Portland Cement Association showed a *gain* in strength of air-entrained concrete over plain concrete when the amount of air was kept down to a certain level, depending on the size of the gravel used. For example, greater strength was obtained on $1\frac{1}{2}$ -inch gravel with less than 3 per cent of air, on $\frac{3}{4}$ -inch gravel with less than $5\frac{1}{2}$ per cent of air, and on $\frac{3}{8}$ -inch gravel with less than $6\frac{1}{2}$ per cent of air.

On two large projects, in Canada, keeping the air down to 3 or 3.5 per cent, and seeing that the slump was not greater than in plain concrete, had resulted in a gain in strength of 7.9 per cent on an average of 56 cylinders of air-entrained concrete compared with 56 of plain concrete. (See Table 4.)

TABLE 4

Cylinders 6" × 12"	Cement : sacks/cu. yd of concrete	Average slump : inches	Compressive strength : lb./sq. in.		Average of air in total mix : per cent
			7 days	28 days	
Job "A"—a dam :					
Average of 28 cys	4.28 plain concrete	3	2,187	3,599	—
Ditto	4.28 with admix.	3	2,444	3,879	3.0
Average of 53 cys	3.5 with admix.	3	1,957	2,966	3.8
Average of 75 cys	4.0 with admix.	3	2,164	3,321	3.8
Job "B"—a bridge :					
Average of 35 cys	6.0 plain concrete	3.35	2,140	3,196	—
Ditto	6.0 with admix.	1.75	2,865	3,984	3.1

NOTES.—In job A, 6-inch crushed gravel had been used in the mix, and all rock larger than $1\frac{1}{4}$ inch screened out to make the cylinders. That was why strength had appeared so high on the cement content of that concrete.

The difference in slump in job B of 1.6 inch in favour of air-entrained concrete should not increase the strength more than 400 lb. per square inch, whereas the increase for air-entrained concrete was nearly 800 lb., a gain of 400 lb. per square inch using the same cement content per cubic yard.

Mr Eldridge thought *Fig. 5* was very misleading, and should not have been published for the very reasons mentioned in the paragraph below it.

Air-entrained concrete could not be compared with plain concrete with the same water/cement ratio, since the main object of air-entraining was to cut out the 10 per cent of excess water and still get better placement. Further, a 6-sack mix with 8 per cent of air would have a yield of 8 per cent more than a plain mix, and the Author was comparing a plain 6-sack mix with a 5.5-sack air-entraining mix with 8 per cent of air and about 20 per cent more water than required to give equal placement.

The Paper indicated a loss of strength by the use of air-entrained concrete. Such loss of strength was not necessary if proper control was exercised by (1) cutting the water down by 10 per cent, (2) using not more than 4 per cent of air, and (3) reducing the amount of sand by 3 per cent.

In the past, mixes were often designed for maximum density, on the theory of an ideal grading that would give minimum voids and maximum density. Such mixes were much too harsh to place, and in practice on structural work the concrete would be badly honeycombed, and would have far from uniform strength or uniform maximum density in place.

It was well known that about 25 per cent excess mortar over that required to fill the voids was necessary to obtain reasonable placement. Even then, without air-entrainment the extra work involved in placing, and the difficulty of ensuring a good stripping job, did not justify the use of plain concrete, especially on structural or architectural work. In fact, so much advantage had been found in using air-entrainment, in the prevention of segregation, obtaining greater uniform strength and density in place, greater workability and ease of placement, reduction in bleeding of slabs and absence of laitance, that no contractor on structural work in Canada would consider using plain concrete.

In short, air-entrained concrete produced at least equal strength with much greater workability, but a rigid method of *quick control* was imperative. Such control had been found in the weight per cubic foot of an average sample of the mix compared with the weight of a cubic foot calculated from absolute volume of each ingredient.

Mr. Eldridge gathered that the Author considered air-meters a more accurate way of measuring the air content. The Paper emphasized that to obtain an accurate figure of the air content with the weight-per-cubic-foot-method it was necessary to take an average sample of the concrete. That was quite true, and until an inspector was reasonably good at taking average samples he should prove his figures by taking three samples from the same mix. If an average was difficult to obtain on a cubic-foot sample, or half a cubic foot, it was still more difficult with 0.25 or 0.2 of a cubic foot, which was the amount used when testing by air-meter.

The percentage of air recorded by an air-meter is three or more times less often accurate than when obtained by the weight-per-cubic-foot method, even if one could rely on the assumption that the air-gauge zero had been checked and the gauge was reading correctly. The air was in the cement paste, and if too much gravel was included in the sample the

percentage of air obtained would be too low, whilst if too much mortar was taken in the sample the air content would be too high. Whenever possible, when using an air-meter, Mr Eldridge weighed the sample in the bowl and checked the gauge reading against the weight per cubic foot.

The Author, on p. 351, mentioned the fact that Walker and Bloem had found that sand between No. 30 and No. 50 A.S.T.M. sieve had a marked effect on the amount of air that could be entrained. Mr Eldridge's experience had been that while the No. 30 to 50 portion was beneficial to air content, the portion between No. 50 and 100 had much greater effect on air entrainment, as also on the workability of plain concrete.

It had been found that a glass gauge with a 3-way valve was the best admix dispenser. The inspector could see the actual amount going into the mixing water. Automatic dispensers were not visual, and should be frequently checked as to actual delivery to the mixer.

A proper test for bleeding was probably the best indicator of non-segregation and workability. A slump cone was often used to determine workability, but it was not all that one could wish for. A simple comparison might be made in the field by driving a bucket of each kind of concrete of exactly the same slump over a rough road and measuring the amount of water coming to the surface.

A simple explanation of the greater flowability of air-entrained concrete might be arrived at by considering the fact that in a heavy-duty engine it was cup grease, not thin oil, that was used to keep the shaft from contact with the bearing metal, to reduce friction. Similarly, the fat plastic mortar in air-entrained concrete kept the gravel pieces apart, and the friction of rock on rock was greatly reduced. A well designed concrete (air-entrained) might be made to flow horizontally at least 25 feet when necessary without segregation of the gravel.

On p. 339 it was stated that air-entrainment had been discovered in the north-eastern States, where certain pavements in which there had been air-entraining cement gave a more durable concrete and greater freedom from surface scaling due to freezing and thawing and also to the addition of calcium chloride. That discovery was, to the best of Mr Eldridge's knowledge, about 1936, 7 years after air-entrained concrete had been used for structural purposes in British Columbia. Its use in mass concrete had not been considered at that time, owing to the comparative ease of placing pavements and other large masses. The climate in Vancouver, where it was first developed, was somewhat similar to the South of England, where freezing and thawing were not a problem as in the eastern and middle-western parts of America.

Mr Eldridge was convinced that the main advantages of air-entrainment were the prevention of segregation, the much greater ease of placement, and the greater uniformity of strength and waterproofness in place. Those were advantages that were world-wide in application—in hot, cold, or moderate climates.

It might be of interest to mention that there were six central mixing plants in the Vancouver area, and that practically all the concrete produced by them was "air-entrained."

In conclusion, there was no doubt that air-entraining in concrete had made a very definite advance in eliminating the serious fluctuations in strength, density, and waterproofness in the average structural concrete job. It was essential, however, that a specification be laid down for admixtures, as to their uniformity, and that a proper measuring device be used, too much variation in measuring admixtures having been responsible in the past for an unnecessarily wide variation in air content. The grading of the sand should also be kept fairly uniform, especially the portion between 48 and 100-mesh, in order to keep the percentage of air constant for the same amount of admix. With plain concrete, sands with less than 12 per cent of 48-mesh gave harsh mixes, but sands with as low as 5 per cent of 48-mesh had been used when no fines were available, by adding double the amount of admixture ordinarily used. That gave about the same amount of air as a well graded sand with half that much admixture.

The Author, in reply to Mr Richards, observed that whilst the explanation was commonly accepted, it was not necessarily true that the reason for the use in America of more workable concrete for road construction than was used in Great Britain was largely to expedite the construction. For example, on an airfield contract at present in progress in England, concrete of low workability was being produced for runway construction at the rate of 15,000 cubic yards per week, which corresponded to 3 miles per week of a roadway 24 feet wide and 8 inches thick. The conditions in airfield work lent themselves more readily to rapid construction than did those of road work, but that rate was believed to be as high as that reached in the United States. Further, more workable concrete was found to make the construction of high-quality concrete roads more difficult, for it tended to flow under the action of surface vibrators in the direction of even a slight crossfall or gradient; that led to poorer riding quality. It was for those reasons that the use of air-entrained concrete was not considered to be highly advantageous for high-quality road construction in Great Britain. The building of high-quality concrete roads, however, represents in Britain one of the very minor uses of cement. The largest uses were in the building and precast-products industries, and it was suggested in the Paper that the manufacturers of certain precast products could benefit by the use of air entrainment as was the case in America.

In contrast to Mr Richards' suggestion that in the United States the air-entraining agent was generally incorporated in the cement, Mr Melville had written as though the usual practice were to add the agent at the mixer. The Author had had no first-hand experience of American methods, but from conversation with a number of American engineers and from

numerous Papers on the subject it appeared that both methods were widely used, and that whilst each had its own merits and demerits neither was gaining universal approval.

Mr Smith's doubts about the use of air-entrained concrete for prestressed work, where high strengths were necessary, were supported by the results shown in *Fig. 7*. Such doubts, however, were based on the results of laboratory tests and it was possible that, because of reduced segregation and greater ease of compaction around congested reinforcement, the use of entrained air might be advantageous in prestressed concrete. There seemed to be insufficient evidence so far to enable a valid judgement to be given. No tests had been made at the Road Research Laboratory on air-entrained mixes richer than 1 : 6, since the work had been confined to mixes in the range suitable for use in roads.

The Author had no comparative figures for the variability of ordinary and air-entrained concrete and such figures would be difficult to obtain. If results from different jobs were compared it would be almost certain that differences would exist which would not depend upon whether or not air-entrained concrete had been used, whilst if experimental work was undertaken to obtain the particular information it was likely that rather more careful control than normal might be exercised and the comparison would not then be a guide to what might be found in practice. Only by continued experience with air-entrained concrete could it be determined whether the ratios of minimum to average strength used in designing ordinary concrete mixes could be used satisfactorily with air-entrained concrete.

Mr Kennard had made the important point that, for estimating the increase in yield, the effective air content was that in excess of what would remain in ordinary concrete after compaction. Inevitably some air voids were present in all concrete and if, in practice, the air content of ordinary concrete was less than one per cent, the compaction might be considered as satisfactory. In laboratory work it was generally possible to obtain a greater degree of compaction than in practice and the left-hand points of the four lines in *Figs 1 (a) and (b)* showed that air contents below one per cent were consistently obtained. The comparison made on p. 349 did not refer to particular results but to the principle. The figures could be taken as referring to ordinary and air-entrained concrete containing 1 per cent and 6 per cent of air respectively.

The Author was grateful to Mr Melville for his contribution concerning the transport of air-entrained concrete. His conclusions would be of wide interest, for little experience in transporting air-entrained concrete had yet been obtained in the United Kingdom.

Mr Fraser had also recorded some very interesting experiences, which were especially useful because they related to practice in Great Britain. In particular, air-entrainment produced a greater increase in workability than would generally be associated with the observed increase in slump.

That was a feature which could only be fully appreciated by working with air-entrained concrete.

The possibility mentioned by Dr Kansoh of the more efficient hydration of cement in the presence of a wetting agent had been suggested by several other authorities. So far as the Author was aware, however, the reality of that effect had still to be proved and he would be interested to hear of any confirmatory data.

The Author could not agree with Dr Kansoh's unqualified statement that the gravimetric method of measuring air content was the most accurate one. It was accurate only if the proportions of the various constituents in the sample being tested, as well as their specific gravities, were accurately known. In the field that information was practically impossible to obtain. Even if weigh-batching equipment was correctly calibrated and carefully used, and the moisture contents of the materials were accurately known, the proportions of the materials in the sample might differ widely from those of the materials entering the mixer. Adhesion of mortar around the blades of the mixer, or the breaking away of adhered material might result in a leaner or a richer batch of concrete; if the concrete was sampled at the discharge chute, the proportions would, in general, depend on whether the sample was taken at the beginning, middle, or end of the discharge of the batch; and if the concrete was sampled after it had been placed in the formwork it was extremely difficult to obtain a sample of representative proportions. Sampling errors, of course, applied to all methods of test, but in using the gravimetric method the effect was two-fold. Not only might the air content of the sample not be representative of the mass but, if the mix proportions were not truly representative, a very considerable error could result when calculating the air content of the sample. Those were some of the very real difficulties associated with the gravimetric method and which, together with the weighing and calculation involved, made it much less suitable than the pressure method for use in the field.

The pressure-type apparatus did, of course, require calibration before use but that had been only briefly referred to in the Paper because it had been dealt with by the various Authors referred to in connexion with the measurement of entrained air. In the usual procedure for calibration, a known volume of air was introduced by placing a cylindrical vessel, with its open end downward, in the container and filling up with water. Pressure was then applied until the indicated air content equalled the volume of air introduced, expressed as a percentage of the volume of the container. The pressure was noted and that pressure was then used for measuring the air content of concrete.

The Author had no evidence that the insulating properties of concrete for heat or sound were affected by entrained air, but it was well known that aerated (lightweight) concrete containing a very much larger proportion of air exhibited much lower thermal conductivity than ordinary

concrete. It was reasonable to assume therefore that entrained air would result in some reduction in thermal conductivity, although the effect would probably be very small.

Little was known of the true tensile strength of concrete, but the flexural strength was reduced for a given water/cement ratio in the same way as the compressive strength was reduced. The reduction was smaller, however—about two-thirds as much by proportion—and it was easier to obtain an increase in flexural strength for a given workability than it was to obtain an increase in compressive strength by entraining air.

Few data were available regarding the effect of entrained air on the bond strength of concrete with ordinary steel reinforcement, but some work had been done in America ²¹, ²² using deformed bars. In general, that work showed a decrease in bond strength with increasing air content, although in some cases a small amount of air had been beneficial. The effect was less, however, than the effect of entrained air on compressive strength.

It was interesting to note that Mr Eldridge considered the gravimetric method of measuring air content to be more accurate than the pressure method. That matter had already been discussed in connexion with Dr Kansoh's comments. The data in Table 4, contributed by Mr Eldridge, were interesting and helped to show that in practice the strength of concrete did not have to be reduced when an air-entraining agent was used. That point was made in the Paper and the Author did not therefore consider that the figures given were misleading.

The Author could not agree with Mr Eldridge's suggestion that a test for bleeding might be the best indicator of non-segregation and workability, for the three properties concerned were not correlated. Most of the work at the Road Research Laboratory concerned mixes of low workability, in which bleeding did not occur. The problems associated with segregation and with workability, however, were as great with those dry mixes as with much wetter mixes. The evidence available indicated that each of the properties of fresh concrete had to be considered separately.

Nor could the Author agree with Mr Eldridge's statement that 25 per cent excess mortar was necessary to obtain reasonable emplacement. Indeed, some excellent concrete structures had recently been built in which gap-graded aggregate of 1½-inch maximum size and little or no excess mortar had been used. In one particular case, a 1 : 8 mix had been used in the vibrated walls of a reinforced-concrete wind tunnel and that had produced the perfectly smooth finish required for the rather special conditions. That type of mix had the added advantage that the shuttering could be removed after 3 to 16 hours, so that the surface could be examined for flaws at an early age.

²¹ H. W. Russell and E. Hognestad, "Effect of entrained air on the bond strength of concrete." *Proc. Highway Res. Bd.*, vol. 28 (1948), p. 195.

²² E. Hognestad and C. P. Siess, "Effect of entrained air on bond between concrete and reinforcing steel." *J. Amer. Concr. Inst.*, vol. 21 (1950), p. 649.

CORRESPONDENCE
on a Paper published in
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Paper No. 5895

" The Control of Concrete Quality " †

by

Leonard John Murdock, M.Sc.(Eng.), Ph.D., A.M.I.C.E.

Correspondence

Professor W. T. Marshall, felt he should challenge the statement made on p. 444 : " Concrete technology is largely neglected in the universities. . . ." The truth of that statement depended on what was meant by the words " concrete technology." If it meant the fundamental principles which should be observed if concrete of suitable strength and workability was to be made, then he disagreed entirely with the Author. Many professors and lecturers had, for a number of years, taught their students the importance of the water/cement ratio, the value of consolidation by vibration, the best types of aggregate to use, the advantages of weigh-batching, etc. Those were the fundamental principles which should be and, in general, were being taught in the universities.

If, on the other hand, Dr Murdock meant that the universities were not teaching their students details of the various pieces of plant required to make good concrete then Professor Marshall submitted that they were doing the right thing by not including that in a university course. Practical details of that nature were best learnt in the field and not in the lecture theatre.

The universities had shown their interest in concrete technology by establishing a Chair in that subject at the Imperial College in 1945. That Chair was largely concerned with post-graduate work, and students were attracted from all universities ; it would be inadvisable from the national viewpoint to have more than one post-graduate Chair in Concrete Technology.

A glance through the Papers printed in the Magazine of Concrete Research would convince Dr Murdock that the universities were certainly doing their share in that aspect of concrete technology.

† *Proc. Instn Civ. Engrs*, Part I, vol. 2, p. 426 (July 1953).

In Professor Marshall's view the people who required education in concrete technology were the men employed on the site. They were the people who mixed and placed the concrete and it was largely a lack of knowledge on their part which was responsible for the mistakes shown in *Figs 3 to 9* in the Paper.

Mr James Ford, of New Plymouth, New Zealand, stated that concrete structures had been built successfully before scientific methods had been applied to the processes of mixing and placing. The soundness of their construction had been demonstrated by the way in which they had resisted demolition in war and major earthquakes. The reason was that the mixing and placing of concrete were an art as well as a science, especially from the mixer to the final deposition.

For an art to exist, there had to be practitioners of that art, and for those practitioners to be successful, they should have a practical understanding of the materials of their art and a mastery of its tools and processes. That was empirical knowledge implying good judgement in its application.

The mixing and placing of concrete were referred to as workmanship, and thereby their true nature was overlooked.

Mr Ford, who had been engaged on construction work at the time that scientific methods of control had been introduced into concrete mixing and placing, had had a fortunate association with an Overseer of Works who had not only appreciated the new methods but also had a thorough understanding of the element of art in the old empirical methods. The Overseer had managed to keep with him a team of key men of long experience who could be relied upon to produce a thoroughly sound finished job, and the structures for which he had been responsible had few if any of the defects shown in the Author's *Figs 3, 4, 6, 7, and 8*. The Overseer had persuaded Mr Ford to spend some time with the team to gain an appreciation of what its members really knew. They could tell by the appearance of the concrete in the mixer what its quality would be like from the point of view of placing and workability; they appreciated the importance of the correct amount of water, and that the limits from too little to too much were very narrow; they could judge quite well, by appearance and handling, the grading of fine aggregate; they had a thorough understanding and appreciation of the correct methods of placing concrete and filling forms.

On large jobs raw workmen had been broken-in as follows; they had been lined up along the formwork and each had had his part of the work explained to him. Later, when the form work had been stripped, they had been lined up in the same positions, and any defects pointed out, so that each man could do better next time.

From that experience Mr Ford had learned that good teams could be trained up for small jobs, but the difficulties were with big jobs, employing large numbers of men. The best that could be done was to place known

good men at strategic points throughout the job and to spend as much time as possible on the job with them—especially at the critical stages.

In the case described above, the sources of trouble had been traced to :—

- (1) Bad feeling amongst members of the placing teams. Their work had been arduous and the steady supply of concrete from the mixer had given them no respite. On adjoining sections they had seen other workmen engaged on tasks not so arduous nor continuous, yet drawing the same rates of pay. The result had been bad workmanship in placing. If at all possible, the more arduous work should be compensated.
- (2) When working on a three-shift basis, the workmen on the mid-night to 8 a.m. shift had not been so alert and bad placing and consolidation could often be traced to that shift. Probably two 10-hour shifts, omitting the hours from midnight to 4 a.m. would be found to give a better quality of workmanship and as great an output.
- (3) Lack of appreciation by junior engineers and by technical assistants of the art or craftsmanship factor as it applied to the job in hand. When conversing with foreman gangers and workmen, they made criticisms of matters and difficulties of which they had no real appreciation. That caused bad feeling which in turn led to poor work. The remedy was to be found in plenty of first-hand experience with the placing gangs. Intimate contact with the work developed that personal judgement (artistic in character) of where the difficulties lay, what the possibilities were, and the proper steps to take. Good leadership played no small part in ensuring that the good concrete which came from the mixer was still good concrete in the finished job.

The foregoing was what Mr G. A. Maunsell had meant¹⁷ when saying “... that first-class scientists seldom make good engineers. The fact is that engineers are essentially craftsmen and the engineering profession should be the great fosterer of the crafts” and “The chief glory of engineering lies in the marshalling of craftsmanship—artistic craftsmanship and creative effort.”

The proper use of vibrators when placing concrete called for intelligence on the part of those using them. Their effect was from the empirical application of the molecular theory of liquids. If a granular mass was vibrated at a suitable frequency and amplitude it would behave as a liquid. That had first been demonstrated by Robert Hooke, of Hooke's Law fame, who had used a container of sand placed on a drum. Beating

¹⁷ G. A. Maunsell. James Forrest Lecture, 1951, “Some Unsolved Problems in Civil Engineering.” *J. Instn Civ. Engrs*, vol 36, p. 361 (Oct. 1951).

the drum had been the only way he could obtain the necessary vibration of the container and its contents.

Solid objects sank and light ones rose in a vibrating granular mass. The application to concreting was that the fine aggregate, under the action of the vibrator, behaved as a liquid in its neighbourhood, and the coarse aggregate tended to sink in it. In practice, the concrete first settled down and, if the vibration was continued, the coarse aggregate sank through the fine, causing segregation.

Concrete could also be made to flow because of that effect, hence the temptation to use the vibrator to move it along the form. The result again was segregation and associated troubles. The effect of the mixing water was probably that of a lubricant. The difficulty was to obtain quantitative results, and the correct frequency and amplitude to cause the concrete to settle readily without segregation. The use of vibration was not a substitute for spading and slicing. Those operations were still necessary to get the concrete into corners and angles in the formwork, and thoroughly compacted around the reinforcement.

Internal vibrators, when running, should not be allowed to come into contact with reinforcement. If that was allowed the reinforcement was shaken and the bond would be destroyed between it and the concrete in which initial set had begun.

The element of craftsmanship in placing concrete was just as necessary with vibrators as without them.

Mr H. C. Erntroy commented that a visual inspection of the points on each of the graphs (*Figs 10 and 11*) would hardly appear to justify the type of mean curve shown, which could almost equally well be taken as a straight line through the origin.

He was in entire agreement with the Author that "... the variability depends on the method of production and the control adopted on the site ..." (p. 439). He appreciated that for many sites it was not possible to obtain sufficient detailed data to determine the degree of control, but nevertheless it seemed a little surprising that the Author had drawn only one mean curve through the points, which were shown separately for weigh- and volume-batching, particularly since they appeared to form two fairly distinct groups.

On the basis of those mean curves the Author had deduced that "... it is clear that for average strengths of more than about 3,000 lb. per square inch, neither the total variation nor the standard deviation shows any indication of increasing with increasing strength." Although it was felt that the results would hardly confirm that contention if points for weigh- and volume-batching were to be considered separately, it might be of interest to note that a theoretical study led to a very similar conclusion.

The variation in the compressive strength of works test cubes resulted from a number of factors. For convenience, those might be considered

in two groups: (a) that produced by the varying rates of hardening of the cement and the inherent variations in the methods of making, curing, and testing cubes; and (b) that arising from the fluctuation in the water/cement ratio. The latter group included all the variations caused by the method of batching, since variations in the proportions of the cement and aggregates and in the grading of the aggregates would probably result in an adjustment of the quantity of water added at the mixer in attempting to maintain a constant degree of workability.

A theoretical analysis of the effect of variations in the water/cement ratio on the compressive strength had been carried out as follows: it was assumed that a specified or desired water/cement ratio, r , could be maintained within a range of $\pm n \times r$ where n would vary with the degree of control being exercised on the job. It was realized that that assumption was probably not justified entirely since it was far more likely that the mix would be wetter than desired rather than too dry. That should not, however, affect the basic conclusion drawn from the analysis. The following values for n were selected:

0.02 (very good control)

0.10 (moderate control)

0.25 (poor control).

From the curve of compressive strength against water/cement ratio shown in Fig. 1 of Road Note No. 4,¹⁸ the strengths at 7 and 28 days corresponding to the upper and lower limits of the variation in water/cement ratio for average values ranging from 0.4 to 1.0 by weight were taken and the range of variation obtained by subtraction. That value for any selected water/cement ratio had been plotted against the corresponding mean strength for that water/cement ratio giving the curves, for the various values of n , shown in Fig. 13. Since the standard deviation was directly proportional to the total variation for a given number of results the vertical axis might also be taken to represent the standard deviation to a different scale.

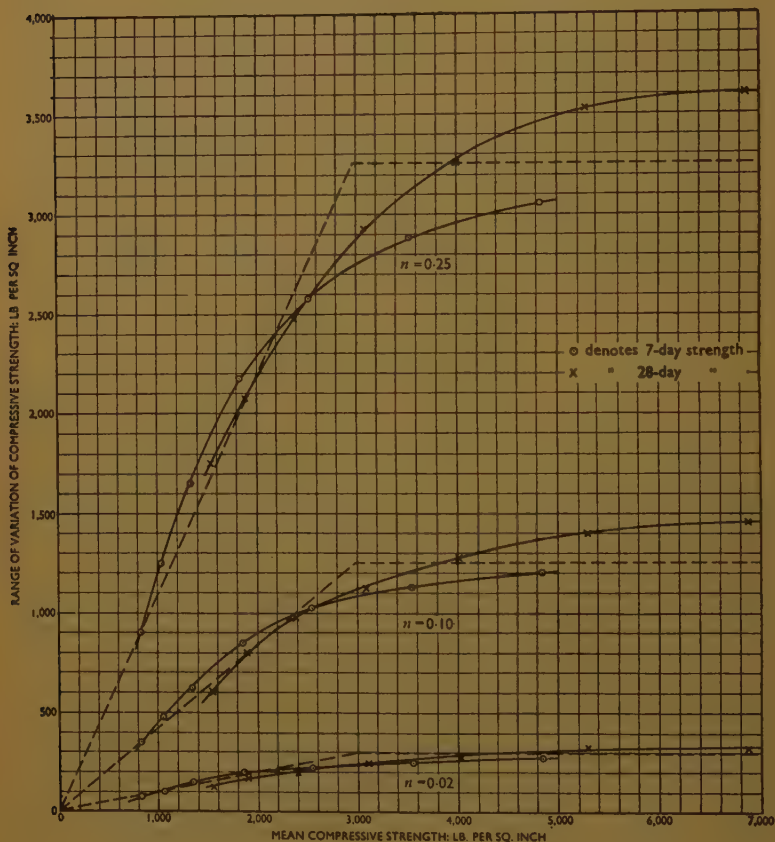
It might be seen that each of those curves could be represented approximately by two straight lines, one through the origin and the other horizontal; that was, in effect, the type of relationship suggested by the Author. It was also of interest to note that the slopes of the lines through the origin were not constant for other ages of test not shown in Fig. 13, nor was the horizontal line at the same level throughout. However, the changeover point in the relationship appeared to be constant at a compressive strength of about 3,000 lb. per square inch and provided some confirmation of the Author's statement quoted previously.

The variations in concrete cube strength considered in group (a) seemed likely to be approximately proportional to the average strength. The

¹⁸ "Design of Concrete Mixes." Road Note No. 4. Rd Res. Lab. H.M.S.O., 1947.

total variation would be made up from the two components (a) and (b), and would therefore not necessarily be of the precise form indicated in *Fig. 13*. It was hoped that further evidence on that subject would become available from work which was being undertaken by the Cement

Fig. 13



THEORETICAL RELATIONSHIP BETWEEN THE RANGE OF VARIATION (ARISING FROM THE FLUCTUATION IN THE WATER/CEMENT RATIO) AND THE MEAN COMPRESSIVE STRENGTH FOR VARIOUS DEGREES OF CONTROL.

& Concrete Association to determine the relationship between the range of variation (or standard deviation) and the average strength of sets of works test cubes for varying methods of control.

Mr A. W. Shilston observed that the standard of manufacture of

in-situ concrete, in Great Britain generally, was low, and practice lagged a long way behind knowledge of mix-design procedure.

He thought that the problem of concrete production was essentially ethical. The will to make the effort (which would involve no great extra cost) to produce a high quality material was lacking, and until that basic defect was remedied, good concrete could never result. There should be a concerted move to instil high ideals in tradesmen and contractors in carrying out their work. A completely new approach was required by many contracting firms and the example, as in all matters, should come from the top. Until commercial motives were more closely allied to professional conceptions, risk of poor concrete production would remain. That could be readily done by ensuring that the boards of civil engineering contracting firms were well represented by chartered civil engineers.

Good concrete required understanding of the complicated nature of the material. Ignorance of the basic principles of concrete manufacture was appalling among foremen, gangers, and leading hands; without further training it was difficult to see how improvement could be made and the introduction of trained concrete technicians was vital. Long experience, usually such an invaluable commodity in civil engineering works, was frequently detrimental to good-quality concrete manufacture, and it would be better if a new community of supervisors and executants were trained, untrammelled by preconceived ideas.

Despite the extensive literature produced, he felt that the obligations associated with prestressed-concrete construction were still not appreciated by some firms embarking on that type of work. For that reason open tendering should be strongly discouraged for both prestressed and normal reinforced-concrete work; invitations to tender should be invited from selected firms of repute. Dr Murdock's photograph of defective in-situ prestressed concrete (*Fig. 4*) confirmed Mr Shilston's own preference for the use of precast prestressed concrete wherever possible.

Dr Murdock had commented on the need for an improved type of concrete-mixer, but Mr Shilston felt that the Author had rather lightly dismissed the use of the pan-type which produced a consistently uniform mix and had many advantages over the more usual types of mixer. It was not unusual, at the present time, for the pan-type of mixer to be specified for works where quality-concrete control was essential and he felt that that trend would continue.

He observed that he had not yet found an accurate and reliable water tank incorporated as an integral feature in a concrete-mixer and he would like to hear whether the Author's experience had been the same. He considered, if positive control was required, that it was preferable for the water tank to be a separate-purpose unit of small horizontal cross-sectional area.

He endorsed Dr Murdock's concluding remarks regarding the need for co-operation on the site between the contractor and the engineer. Fewer

subjects were more likely to cause bad feelings between the contractor and the resident engineer's staff than concrete control—particularly on the smaller works—since the standard of concrete varied frequently in direct proportion to the persistence and tenacity of the engineer's representative. A positive effort on the part of the contractor, to supervise the operations of his workmen, was, therefore essential if good concrete and harmonious relations were to be obtained.

Mr F. A. Sharman remarked that a Stationery Office publication¹⁹ could be produced to support the view that with "good" control, the target strength should be 154 per cent of minimum, and that "fair" control required 172 per cent of minimum. The relation between average strength and variation was such that with an average strength below 4,000 lb. per square inch the minimum was expected to be below half the average. Against that, records existed of jobs (apart from such special cases as London Airport) on which much better results had been achieved. For example, the weigh-batched concrete at Port Talbot gave minima of 2,600 and 2,800 lb. per square inch and averages of 3,744 and 3,582 lb. per square inch respectively.²⁰

When Dr Murdock's collection of results was examined, it might be expected that a "normal" level of control would be indicated, with a few backward or exceptionally difficult sites showing greater variation, and still fewer jobs of the London Airport variety producing the smallest variations possible. The actual state of affairs was quite different; variations were scattered in an absolutely random distribution between the best and the worst. A distinction had been made between sites where control was known to be good, and where the degree of control was unknown, but unluckily the average strength of all but one of the former group lies in the very limited range between 5,100 and 6,300 lb. per square inch, whereas the latter covered average strengths between 2,345 and 7,790 lb. per square inch. Mr Sharman asked, was it not likely that if more sites with known good control (and a consequent lower tendency in their standard deviations) had been observed in the lower average-strength classes, a different distribution of points might have been seen in *Fig. 11*?

In the Synopsis it was stated that statistical analyses were of very little value for day-to-day control, and in the Tables and graphs dealing with variation, the maximum and minimum recorded strengths were given almost equal prominence with the coefficient of variation. That might seem to encourage the point of view, sometimes held, that the range of values obtained was the simple and satisfactory thing to note, whilst the analysis of variation could be left to people who enjoyed such things. Would not the Author agree that the interpretation of test results on a

¹⁹ A. R. Collins, "Effect of Batching Errors on the Uniformity of Concrete." Road Note No. 3. D.S.I.R. Rd Res. Lab. H.M.S.O., 1947.

²⁰ Donald Lombard and A. A. Wright, "New Steelworks, Port Talbot: Construction of Works." Works Construction Paper No. 17, Instn Civ. Engrs, 1950.

material such as concrete was essentially a problem in which the statistical techniques of sampling and estimating error were very valuable? Was there any better way of predicting and controlling results from a given concrete-producing set-up than the production of a large number of cubes in a short trial period, during which observable variables were correlated with cube-strength variations? As for the recording of the highest and lowest values, even when the number of results was very large those quantities remained a very unsatisfactory indication of variation, since they, to such a great extent, depended on chance.

If the standard variation was taken as the best measure of control achieved, it should be possible to make some valid deductions from the results plotted in *Fig. 11*. The Author had drawn a "tentative mean curve" and from it he had deduced "variations in strength which must be expected in site concrete" (Table 8). The level of concrete technology varied so widely, at the present time, that to draw a line through the middle of good, bad, and indifferent results and to base on that some sort of standard level of control, however tentative, might appear somewhat fatalistic. If the points representing sites with good control were distinguished from the others it would be seen that there was insufficient information to draw any conclusions about the relationship between strength and variation. Was there any reason to suppose that the dotted extension of the curve passed through the origin? Surely any functional relationship between the two variables became discontinuous before zero strength was reached. The part of the curve below 3,000 lb. per square inch was of considerable importance. In the absence of further results in that region might it not have been better to leave that portion of the curve as a query?

The Author spoke of the need for further research, indisputably needed, but his view on the immediate method and objective of such research would be welcome. Would the Author consider that testing and sampling techniques, and the recording of results were generally sufficiently advanced for the comparison of results between one site and another to be relied on? Surely the general aim of the study should be to establish standards of control to which everyone concerned in the specification and production of concrete could refer?

Mr B. E. P. E. Roberts remarked that Dr Murdock referring to the variability of concrete strength, gave a number of figures to illustrate the amount of variation which might be expected under various conditions.

The results of tests from London Airport (Table 2) showed the low degree of variation which, in certain circumstances, it was possible to achieve. That remark applied particularly to the figures for the year 1950.

It was noteworthy that the standard deviations for the years 1945 to 1949 had shown very little change, indicating that it was possible to make concrete for a long period with almost a uniform variability. That was consistent with Mr Roberts's observations.

A striking point about Table 2 was the considerable variation in the monthly standard deviations for the year 1950. The number of cubes tested each month was not stated, but as the total for the year had been more than 4,000 it seemed unlikely that less than, say, 150 had been tested in any one month. On that assumption the changes in standard deviation had been much greater than would have been expected from sampling fluctuations. Could the Author give any explanation for that?

It was a pity that cement had been tested in sufficient quantity to make comparison possible only in 1945 and 1946, but, so far as those figures went, they were very interesting.

The standard deviations of the cement tests (deduced from averages and coefficients of variation in Table 2) had been 394 and 454 lb. per square inch for 1945 and 1946 respectively; the standard deviations of the concrete tests for those two years had been 527 and 487 lb. per square inch; showing that there had been greater variation in the cement tests in 1946, and greater variation in the concrete tests in 1945. That suggested that changes in water/cement ratio had had more effect than changes in cement quality on the amount of variation in the concrete tests.

In Table 7 comparison was made between weigh-batching and volume-batching by considering overall distributions, each involving test results from a number of sites. An alternative method was to set out the standard deviations listed in Tables 3, 4, 5, and 6 in ascending order, comparing them by inspection as shown in Table 9.

TABLE 9.—COMPARISON OF STANDARD DEVIATIONS

Table 3 (Sites 1 to 4, and 6)	Table 4 (Sites 10 to 12, 14, 17, and 18)	Table 5 (Sites 33/1, 33/2, and 34)	Table 6 (Sites 40, 41, and 49 to 51)
Weigh-batching, good control	Weigh-batching, control unknown	Volume-batching, good control	Volume-batching, control unknown
28-day tests : lb. per sq. in.	28-day tests : lb. per sq. in.	7-day tests : lb. per sq. in.	28-day tests : lb. per sq. in.
(1)	(2)	(3)	(4)
108	670	471	471
526	782	481	729
526	797	556	740
604	838		765
674	890		987
	899		

Estimates of standard deviation based on small numbers of tests were likely to be unreliable and Table 9 included only those sites for which one hundred or more test results had been available.

In comparing estimates of standard deviations it had to be borne in mind that such estimates were subject to sampling fluctuations, and small differences were not significant. A further complication arose, because the figures in column 3 were estimated from 7-day tests, and those in the other columns from 28-day tests. It seemed probable that the variation in concrete strength was somewhat greater at 28 days than at 7 days, and the figures in column 3 were less than they would be if calculated from 28-day tests.

Columns 2 and 4 were records from sites where the control conditions were unknown. Control, therefore, might have been good on some of those sites, and the first figure in each of those columns could evidently be classified under columns 1 and 3 as indicated by the arrows.

The comparison between columns 1 and 2, and between columns 3 and 4, showed clearly the considerable improvement in uniformity which resulted from good control.

In the conditions represented by columns 2 and 4 there appeared to be no difference between weigh-batching and volume-batching.

The comparison between columns 1 and 3 was complicated both by the few results in column 3, and by the difficulty in comparing statistics of 7- and 28-day tests. Making appropriate allowance for that, it appeared that where conditions were suitable for a really high degree of control, weigh-batching gave the more uniform concrete; where such high degree of control was not practicable, however, volume-batching would probably give equally good results. It would be interesting to have the Author's views on the comparison.

It would be noted that the results obtained with continuous mixers had not been excluded.

The phrase "good control" suggested, at first glance, efficiency in plant maintenance, and supervision of concreting, etc., but Mr Roberts thought that the Author had in mind a wider meaning. The amount of variation in concrete strength was dependent on a number of factors, some of which were generally not within the control of the engineers and other personnel on the site, such as :—

- (1) Necessary variations in workability to suit the different concreting operations on the site.
- (2) Variations in grading of the aggregates necessitating changes in water/cement ratio to maintain the required workability.
- (3) Variations in cement quality.
- (4) Richness of mix : the workability of rich mixes was less affected by changes in grading than that of lean mixes.
- (5) Method of compacting : when powerful mechanical means of compacting were used changes of workability could be tolerated within wider limits than when hand compacting was employed.

- (6) Accidental changes in water/cement ratio, both direct and resulting from accidental changes in proportioning: they were dependent on proper maintenance of plant, supervision of batching and mixing, etc.

The proportion of the total variation brought about by (1) to (5), which might be considerable, would vary from site to site. A high standard deviation was not necessarily a reflection on either the personnel or the choice of plant.

That consideration, which was taken into account by the Author in Table 8, suggested that any form of comparison of Tables 3, 4, 5, and 6 should be accepted with some reserve.

The Author had pointed that out, emphasizing the need for further research. What was needed was the comparison of a large number of different works for which information on all the different factors was available.

The amount of variation actually observed on any site was, to some extent, dependent on the number of test cubes taken. If a specified low test occurred in the long run once in every hundred cubes tested, the chance of its occurrence at least once in twenty cube tests was about two in eleven; if, however, two hundred cubes were tested, the chance of the specified low test being observed increased to nine in ten. If only a few cubes were tested the variation would appear to be low, and the variation in the strength of the concrete in the works might be much greater than indicated by the test cubes.

Table 8, which the Author said was tentative, summarized the information given in the Paper in the form of recommendations for practice.

The permissible variations (for average strength of more than 3,000 lb. per square inch at 28 days) were given as independent of the average strength. That was in agreement with Mr Roberts's observations. Previously such limits had been suggested as proportional to the average strength.

The Author stated that 2 or 3 per cent of tests might be expected to fall outside the lower limits, and, if that was not acceptable, greater variations should be allowed. A reasonable interpretation of a minimum strength specification would seem to be that not more than 1 per cent of the tests should fall below the specified minimum. Two or 3 per cent seemed to be rather on the high side.

If it was assumed that 28-day test cubes were distributed normally (in the statistical sense), the limits of variation suggested in Table 8 would correspond to the standard deviations in column 2 (Table 10), on the assumption that $2\frac{1}{2}$ per cent of tests would fall below the specified minimum.

Those standard deviations corresponded well with the test results from concrete works given in the Paper, but Mr Roberts believed that in some works, where a relatively lean concrete was made for hand compacting,

TABLE 10

Variation in Table 8	Standard deviation	Variation if only 1 per cent of tests fall below the specified minimum
lb. per sq. in.	lb. per sq. in.	lb. per sq. in.
(1)	(2)	(3)
1,000	500	1,200
1,200	600	1,400
1,500	750	1,750
1,800	900	2,100

and particularly where considerable variation in the grading of fine aggregate was unavoidable, standard deviations well above 600 lb. per square inch might be recorded, even with weigh-batching equipment and good supervision.

Column 3, in Table 10, indicated the corresponding variation to be allowed for if only 1 per cent of the results were to fall below the lower limits.

Such Tables were principally of use in fixing the average strength to be aimed at when designing a mix, and they gave the amount by which the average had to be greater than the specified minimum.

For checking conformity with a specification, Tables such as those would not be found sufficiently sensitive, and only departures of considerable magnitude would, in general, be detected.

A somewhat more sensitive method could be obtained by taking averages of six consecutive tests and plotting them on a control chart for means, as described in B.S.600 R.

Although reduction of variation in strength was on the whole a good aim, it might be wise to avoid too much emphasis on that aspect of control.

He believed that in certain circumstances a measure of uniformity could be obtained by using a relatively high water/cement ratio, the concrete having an unnecessarily high workability. Greater variability and a lower average water/cement ratio might well result in a concrete conforming to the same minimum strength requirement, having a higher average strength and better resistance to deterioration.

Mr G. Stevenson observed that there appeared to be a certain lack of realization, on the part of engineers preparing contracts involving a large percentage of concrete work, as to the necessity for up-to-date specifications. The result was that the quality control necessary for good concrete was neglected and left entirely to the discretion of the contractor. The average contractor left the making, placing, etc., of concrete to a foreman who might or might not be aware of what constituted good concrete. Even if given a new mixer, accurate water gauge,

weigh-batcher, adequate storage facilities, etc., he would still require to be supervised by a competent clerk of works or inspector who would check consistencies, examine cement, test weigh-batcher at intervals, make test cubes, ensure adequate curing, etc.

The recommendations given for achieving good results were certainly those which should be adopted as a whole or in part by all engineers; Mr Stevenson had adopted methods (3) and (4) as being the only ones possible under the existing specifications for average-sized contracts. There was no doubt that foremen were interested in producing good concrete and it was a pity that contractors could not be persuaded to send those men on the special courses which were organized for their benefit to instruct them in concrete technology.

He found that the only satisfactory way to ensure that aggregates were kept clean was to insist on the construction of a weak-concrete apron if no other clean area was available.

The use of a weigh-batcher should give more consistent results than volume-batching, but control was normally more important in the former case as the machine needed regular checking. On the whole the latter method might not give the high results obtainable with weigh-batching but the average strength of the concrete could be maintained at the correct level with less supervision.

The use of various patent bolts, cast into concrete for vertical faces, appeared to be the best solution to many of the faults shown in *Fig. 5*. Two or three types of steel formwork incorporated that system; only badly bent plates or walings, or putting tension on the bolts before the concrete has attained a reasonable strength, could lead to poor finish on the resultant face.

Mr Stevenson felt that the observation that "The time interval between mixing and placing is not of serious importance . . .," although qualified by certain provisions, was somewhat sweeping as other factors were involved besides those mentioned. Transportation methods, for example, dumpers, monorail, cranes, and skips, type of cement being used, and the method of compaction of the concrete were all factors contributing to the rate of setting. Further recent experience had shown that an aggregate such as limestone was conducive to quick setting. Under normal circumstances, therefore, there should be no relaxation in the specification that the concrete should be placed as soon after mixing as possible. That was especially necessary where concrete was being made with the minimum water/cement ratio when vibration was the method of compaction.

From experiments it had been found that it was almost impossible to over-vibrate a mix in which the water/cement ratio approached a minimum. Would the Author give his own experience in that connexion?

Curing of concrete was an operation to which little attention was given apparently as a result of lack of appreciation of its importance and of the

organization needed to perform it effectively. Particular supervision should be paid to that operation, since it was normally simple to cure horizontal surfaces ; but wall faces, columns, and the like, required draping with hessian or some other moisture-retaining material. It was often impracticable to cure for the specified time, on account of the method of construction adopted, but that difficulty could normally be foreseen and allowances made. More thought should be given by engineers in preparing specifications so that the contractor or builder could not neglect that important operation. It would be interesting to see comparable results for adequately cured concrete and air-cured concrete.

As a matter of interest, Mr Stevenson gave two sets of results in Table 11 which were comparable since they were taken during the same period on two different contracts. The aggregates were similar and the only difference occurred in the cements, which were from two distinct works. On site D the proportioning of aggregates was done by weight whilst on site R, it was by volume.

TABLE 11

Site *	No. of cubes tested	Compressive strength : lb. per sq. in.			Age of cubes : days
		Highest	Lowest	Average	
D	24	4,600	2,860 †	3,650	7
	12	5,730	3,980 †	5,230	28
R	12	4,960	3,360	4,470	7
	5	6,830	5,650	6,190	28

* Mix proportion :—1 : 1½ : 3.

† Strengths obtained after using a proprietary brand of concrete hardener, eventually proved injurious. The strengths specified as required on the works, were those given in B.S. Code of Practice CP 114 (1948). Supervision was by a full-time Clerk of Works.

The Author, in reply, stated that he had made the statement on p. 444, which had been challenged by Professor Marshall, after he had put questions to about a dozen graduates who had come to him straight from Universities. He had found that more than three-quarters of them did not understand the importance of water/cement ratio, the influence of aggregates on workability, and other fundamental principles as suggested by Professor Marshall. They were usually better versed in other aspects of concrete technology, particularly in connexion with matters of structural design. It was that lack of balance between the two aspects of concrete technology which the Author was criticizing.

The Author agreed with Professor Marshall that the men employed on the site required education in concrete technology, but it was also essential

that the engineer and others responsible for supervision, were able to give the tuition which the men needed. Attempts had been made by more than one Association to arrange evening lectures in concrete production at various centres, and one of the greatest difficulties had been the lack of good lecturers.

Mr Ford had stated that concrete structures had been built successfully before scientific methods had been applied to the process of mixing and placing, but it was only fair to point out that there had been failures too. The failures shown in *Figs 6 and 9* had occurred in structures built 20 years or so ago. The Author believed that the introduction of scientific methods in recent years, had brought about an immeasurable improvement in the general quality of concrete placed in Great Britain. The results of that improvement would be shown by a marked reduction in the number of structures showing signs of failure, such as spalling of concrete from the reinforcement, disintegration by frost, erosion, failure along construction joints, and so on.

The Author agreed with Mr Ford on the need to build up teams of key men of long experience, who could be relied upon to produce a thoroughly sound finished job, and he agreed with the comments Mr Ford had made on the training of such teams.

The Author was indebted to Mr Erntroy for his account of a theoretical study of the cube-strength results, which had led to a similar conclusion to his own, concerning the change-over point at a compressive strength of about 3,000 lb. per square inch. Mr Erntroy and Mr Sharman had referred to *Figs 10 and 11*. When drawing those curves, the Author had had available the results from a considerable number of sites where only a few cubes had been made, but their inclusion in detail in the Paper was not justified. He had, however, used them collectively when attempting to show the general trend. The scatter was considerable, but he believed it was useful to know whether a batch of results was better or worse than average.

Mr Shilston had made interesting comments on the production of good concrete, which appeared somewhat at variance with the views expressed by Mr Ford. Mr Shilston had remarked that the standard of manufacture of in-situ concrete in Great Britain generally, was low, but the Author would not entirely agree with that, particularly in the case of the more important structures. In that connexion, it might be interesting to note that the photograph, *Fig. 4*, had been taken on the Continent, where the Author had found the same faults in concrete production, and the same types of failure as in Great Britain.

The Author had not discussed the pan-type of mixer in detail, because he was not sure that the segregation of materials was any less than that in the drum-type of mixer. When using stiff mixes in a pan mixer he had found some lack of uniformity, the stones being thrown to the sides or centre, according to the direction of rotation of the mixing star.

The Author thought that the vertical-syphon type of water tank was reasonably satisfactory, but had found it necessary to calibrate each tank individually. That was done with all mixers immediately after overhaul. The Author believed that the whole question of water measurement, and of mixer design for stiff mixes, needed consideration.

Referring to Mr Sharman's comments, the Author had been connected with the concrete at Port Talbot, and had included some of the results in Tables 3 and 5. That particular work had involved wide variations in the dimensions of the concrete members, and the production of concrete at varying rates from several different types of mixer. The Author did not think Mr Sharman's suggestion of producing a large number of cubes in a short trial period was likely to prove very successful in practice, since the quality control was likely to be at its best during that period and might not be maintained. There were long term variations, for instance, owing to changes in cement quality, and the grading of aggregates, particularly the sand, was likely to vary from month to month. The Author had arranged for statistical analyses to be made during the course of a number of jobs, in order to obtain information for future use, but had not found such analyses of sufficient value in day-to-day control to justify the work entailed. The simple question of whether a cube passed the minimum, or exceeded the maximum, could be understood by all on the site.

Mr Sharman had asked for some suggestions for further research, but the Author found it very difficult to suggest particular items from the many problems which existed. The Author felt that a better method of measuring workability and a fuller understanding of the term workability was one essential. Following workability, came the question of concrete-mix design, and, although the method suggested in Road Note No. 4, was a first step, it was still not possible to predict with sufficient accuracy, the workability and strength obtainable with any particular aggregate, cement, and water/cement ratio. The Author had found, in recent work, that the proportion of the finer particles of sand in the mix was of much greater importance than that of the larger sizes; also that the type of cement used had a marked effect on workability. There was the need for a method of assessing any tendency of a concrete mix to segregate. Again, there was the influence of the type of aggregate on the compressive strength of concrete; the Author had found some indication that certain limestones tended to give higher strengths than gravel aggregates. The testing of cubes was not entirely satisfactory, serving only as a guide to the potential quality of the concrete in the structure, and further development of such methods as the ultrasonic sound method to the stage when it could be taken into general use, was very desirable. Until a better understanding of the points mentioned above and many others were obtained, the Author felt it would be difficult to establish standards of control which could be other than tentative.

The basis of control suggested in Table 8 was founded, not only on the

cube strengths given in Table 3-6, but on general experience in controlling quality on many sites large and small. Knowing site conditions, the type of equipment used, and the effort likely to be made by the supervisory staff in controlling quality, it had been found possible to predict with reasonable accuracy, the average strength and range in cube strength likely to occur.

Various other points raised by Mr Sharman were covered in the reply to Mr Erntroy.

In answer to Mr Roberts, the Author stated that he too, had been puzzled by the monthly standard deviations for the year 1950 given in Table 2. He could give no definite reason, but he thought that a number of factors, such as periodic inaccuracies in weigh-batching, changes in sand grading, which would affect the water/cement ratio required to give the necessary workability for the vibrating machines, and changes in cement quality, were factors which had caused greater variability over some periods than others. Mr Roberts had given an interesting alternative method of comparing the standard deviations in Table 9, and the Author agreed with the deductions he had made.

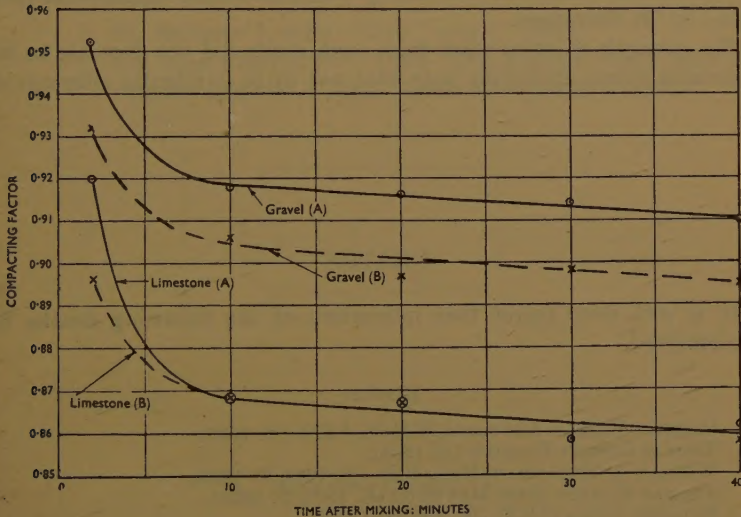
With regard to the comparison between weigh-batching and volume-batching, the Author was of the opinion that weigh-batching gave the more uniform concrete when a high degree of control was exercised. Without careful maintenance and checking of the weighing equipment, however, volume-batching by well-proportioned containers for measurement, was equally good or, perhaps, better. That was one of the reasons why weigh-batching was found to give better results on the larger concreting works where central batching set-ups were employed with proper mechanical maintenance, than on small sites where maintenance was carried out by the concreting gang or others between visits of a fitter.

Mr Roberts had suggested that 2 or 3 per cent of tests falling below the specified minimum was rather on the high side. The Author had noticed from some of the records made available to him, that the employing authority had been more lenient than the figure he had suggested. He had not given any comparison in Tables 3-6 with specified minimum strengths, because he had felt that the varying conditions such a comparison would have little value. Further, with some of the mixes which had been specified, it could be shown that it was not possible to achieve the minimum strength, even allowing for failure of up to 5 per cent of the cubes. He felt that if the greater variation given by Mr Roberts in Table 10 were allowed for in assessing the minimum strength for specification purposes, there would still be a tendency to tolerate failure of 2 or 3 per cent of the cubes. He agreed with Mr Roberts that rich mixes were easier to handle than lean ones and that too much emphasis should not be placed on the saving of cement, since the greater degree of control necessary for lean mixes and the extra costs involved might well outweigh the saving achieved, while not producing a better structure.

The Author agreed with many of Mr Stevenson's comments and was

very glad that the question of curing had been raised, as he, too, felt it was often neglected. It was quite true that it was almost impossible to over-vibrate a mix in which the water/cement ratio approached the minimum. However, one of the chief problems in vibrating stiffer mixes was the occurrence of air bubbles at the surface, and, although the Author had seen and tried various methods which had been advocated, he had not found one which he could confidently recommend as ensuring freedom

Fig. 14



WORKABILITY OF CONCRETE WITH GRAVEL AND LIMESTONE AGGREGATES

Mix in each case was 1:6 (cement:aggregate) and river sand was used for all mixes.

All mixes : effective water/cement ratio.

Allowance for absorption of aggregate was made by:—

(A) Extra water added at time of mixing (simulating the use of a dry aggregate).

(B) Sufficient water added to dry aggregate to give saturation $\frac{1}{2}$ hour before mixing.

Mixing and curing temperature: $64^{\circ}\text{F.} \pm 2^{\circ}$.

Evaporation was presented before and after mixing.

from this blemish. He was conducting a series of experiments in an attempt to obtain information from which definite instructions could be given to the site operatives.

The Author had deliberately included the sentence that the time interval between mixing and placing was not of serious importance, with certain provisions. The statement was based on observations at London

Airport and some recent experimental work, in which he had been assessing the changes in workability of concrete with time after mixing. Some of the results for gravel and limestone aggregates were given in *Fig. 14*. As would be seen, there was, in each case, a marked change in the workability during the first 5 minutes or so after mixing. That change was greater when using dry limestone than when using previously saturated limestone, because, the Author believed, of the rapid initial absorption of water by the limestone. The limestone was of the hard mountain limestone type usually used dry from the crusher, and the Author thought that it was that change in workability which had been described as quick setting by Mr Stevenson.

The strength of cubes made from such mixes did not decrease to any appreciable extent during the tests, that was, up to 40 minutes after mixing.

DEATHS

It is with deep regret that intimation of the following deaths has been received.

Members

- CHARLES BUXTON ANDERSON, C.M.G., I.S.O. (E. 1925).
 THOMAS GEORGE BOCKING (E. 1934).
 WILLIAM ARTHUR GRAY, M.C., M.Sc. (E. 1916, T. 1936).
 THOMAS EDWARD HETT HEYWOOD (E. 1903, T. 1923).
 WILLIAM HOOD, O.B.E. (E. 1927, T. 1946).
 Col. GEORGE ERIC HOWORTH, O.B.E., M.C., B.Sc. (E. 1915, T. 1926).
 GEORGE MATTHEWS HUTTON (E. 1919, T. 1950).
 JAMES SINCLAIR JACKSON, M.C., B.A., B.Sc. (E. 1917, T. 1930).
 WALLACE STRICKLAND LAKE (E. 1905, T. 1920).
 JAMES LOWE (E. 1902, T. 1921).
 WILLIAM GIBB McCracken, B.Sc. (E. 1920, T. 1933).
 JOSEPH LESLIE MUSGRAVE (E. 1943).

Associate Members

- HARRY LATOUR BAZALGETTE (E. 1911).
 ARTHUR ERNEST DARBY (E. 1908).
 GEORGE REINHARD HARRISON (E. 1923).
 HUBERT HAUGHTON (E. 1926).
 RALPH HILTON (E. 1946).
 FREDERIC HORSPOOL (E. 1910).
 JAMES LEE (E. 1935).
 JOHN MCGROBY (E. 1909).
 ROBERT WILFORD PEAKE, B.Sc.(Eng.) (E. 1923).
 LESLIE PYKE (E. 1893).
 WILLIAM HARLEY SAXTON (E. 1918).
 RICHARD DEGGE SITWELL (E. 1911).
 ROBERT EDGAR WALSH, M.C., M.A. (E. 1921).

CORRIGENDA

Proceedings, Part I, January 1954

p. 66, footnote †, *after* " 15th " insert " May "

p. 106, lines 4 and 5, *for* " Loch Sloy, Tummel-Garry, and Shira
power schemes," *read* " Loch Sloy power scheme,"

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